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(INSTITUTED 1852.)

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CONTENTS.

PAPERS.

NO.		PAGE
878	**HISTORY OF THE PENNSYLVANIA AVENUE SUBWAY, PHILADELPHIA, AND SEWER CONSTRUCTION CONNECTED THEREWITH.**	
	By **George S. Webster and Samuel Tobias Wagner**	1
	Discussion on Paper No. 878:	
	By GEORGE S. WEBSTER and SAMUEL TOBIAS WAGNER	31
879	**EXPERIMENTS ON THE FLOW OF WATER IN THE SIX-FOOT STEEL AND WOOD PIPE LINE OF THE PIONEER ELECTRIC POWER COMPANY, AT OGDEN, UTAH. SECOND SERIES.**	
	By **Charles D. Marx, Charles B. Wing and Leander M. Hoskins**	34
	Discussion on Paper No. 879:	
	By E. KUICHLING	55
	G. C. WHIPPLE	59
	J. C. MEEM	60
	G. S. WILLIAMS	61
	GEORGE W. RAFTER	67
	GEORGE H. FENKELL	73
	D. C. HENNY	80
	CHARLES W. SHERMAN	84
	DESMOND FITZGERALD	86
	CHARLES D. MARX, CHARLES B. WING and LEANDER M. HOSKINS	88
880	**RECENT STADIA TOPOGRAPHIC SURVEYS: NOTES RELATING TO METHODS AND COST.**	
	By **William B. Landreth**	92
	Discussion on Paper No. 880:	
	By EMILE LOW	100
	JOHN F. WALLACE	103
	HENRY B. MAGOR	103
	R. S. BUCK	105
	HENRY GOLDMARK	105
	A. J. HIMES	105
	R. A. MACGREGOR	112
	KENNETH ALLEN	113
	WAGER FISHER	114
	G. L. CHRISTIAN	115
	GEORGE W. RAFTER	116
	WILLIAM B. LANDRETH	117
881	**THE NINETY-SIXTH STREET POWER STATION OF THE METROPOLITAN STREET RAILWAY COMPANY, OF NEW YORK CITY.**	
	By **L. G. Montony**	119

19663

NO.		PAGE.
	Discussion on Paper No. 881:	
	By F. L. Averill	143
	Albert Carr	144
	W. A. Low	145
	Werner Boecklin, Jr.	146
	J. D. Kent	147
	L. G. Montony	147
882	**IRRIGATION STUDIES.**	
	By Elwood Mead	149
	Discussion on Paper No. 882:	
	By Mansfield Merriman	176
	Rudolph Hering	176
	A. D. Foote	177
	Elwood Mead	179
883	**A PROPOSED METHOD FOR THE PRESERVATION OF TIMBER.**	
	By F. A. Kummer	181
	Discussion on Paper No. 883:	
	By Henry G. Prout	194
	J. H. Deghuée	195
	F. W. Skinner	195
	Henry Goldmark	197
	Robert Cartwright	198
	V. G. Bogue	198
	Joseph O. Osgood	198
	Samuel Whinery	200
	Horace J. Howe	201
	William Moncure	202
	J. I. Boggs	203
	Walter W. Curtis	204
	R. Montfort	208
	C. F. W. Felt	210
	George S. Valentine	210
	F. A. Kummer	211
884	**ON THE FLOW OF WATER OVER DAMS.**	
	By George W. Rafter	220
	Discussion on Paper No. 884:	
	By George Y. Wisner	315
	Gardner S. Williams	316
	E. Kuichling	335
	E. A. Fuertes	339
	Robert E. Horton	340
	Walter C. Parmley	346
	George T. Nelles	359
	George W. Rafter	383
885	**FILTRATION OF WATER FOR PUBLIC USE. AN INFORMAL DISCUSSION.**	
	By Rudolph Hering	399
	George F. Deacon	405
	George W. Fuller	408
	Samuel Rideal	417
	Henry Davey	419
	J. N. Greene	420
	Ad. Kemna	420
	K. E. Hilgard	425

NO.		PAGE.
	Walter Hunter	430
	Philip H. Palmer	431
	E. Perrett	433
	Andrew Johnston	433
	H. Alfred Roechling	436
	Nicholas Simin	438
	Rudolph Hering	443
886	**HEIGHT OF BUILDINGS. AN INFORMAL DISCUSSION.**	
	By Corydon T. Purdy	449
	Reginald Pelham Bolton	454
	Robert W. Hunt	457
	W. E. Riley	458
	H. de B. Parsons	462
	R. J. Gifford Read	468
	Oscar J. Kirby	468
	John F. Wallace	468
	Maurice M. Sloan	470
	William Copeland Furber	472
887	**RECENT PRACTICE IN RAILS. AN INFORMAL DISCUSSION.**	
	By Robert W. Hunt	475
	Albert Ladd Colby	491
	William R. Webster	496
	Sir Lowthian Bell	499
	J. D. Smelt	502

PLATES.

PLATE.		PAPER.	PAGE.
I.	Subway Sewer Construction, Philadelphia, Open Cut and Tunnel.	878	17
II.	Subway Sewer Construction, Philadelphia, Tunnels	878	19
III.	Subway Sewer Construction, Philadelphia, Profiles	878	23
IV.	Subway Sewer Construction, Philadelphia, Profiles	878	29
V.	Manometers and Difference Gauges on Ogden Conduit	879	39
VI.	Table of Experimental Data for Ogden Experiments	879	41
VII.	Hydrographic Curves for Steel Pipe, Ogden Conduit	879	43
VIII.	Hydrographic Curves for Wood Pipe, Ogden Conduit	879	45
IX.	Hydrographic Diagrams, Ogden Experiments	879	63
X.	Foundations, etc., Ninety-sixth Street Power Station	881	135
XI.	Chimney, Building, etc., Ninety-sixth Street Power Station	881	137
XII.	Engine Room, etc., Ninety-sixth Street Power Station	881	141
XIII.	Register for Weir Measurements	882	161
XIV.	Lower end of Channel, Cornell University Hydraulic Laboratory.	884	301
XV.	Diagrams of Flow of Water over Weirs and Dams	884	331
XVI.	Diagram of Flow of Water over Dams	884	341
XVII.	Views of Lower New York and The Waldorf-Astoria Hotel	886	451
XVIII.	The Old Colony Building, Chicago	886	453
XIX.	The North American Trust Building, New York City	886	455
XX.	Some Typical High Buildings in New York City	886	457
XXI.	Views from American Surety Building, and of Manhattan Savings' Fire	886	463
XXII.	Windsor Hotel Fire	886	465
XXIII.	Home Life Building Fire	886	467
XXIV.	Home Life Building Fire	886	469

Vol. XLIV. DECEMBER. 1900.

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No. 878.

HISTORY OF THE PENNSYLVANIA AVENUE SUBWAY, PHILADELPHIA, AND SEWER CONSTRUCTION CONNECTED THEREWITH.

By George S. Webster and Samuel Tobias Wagner, Members, Am. Soc. C. E.

Presented March 21st, 1900.

WITH DISCUSSION.

It is the purpose of this paper to present the more interesting features of the history of the work of abolishing grade crossings on Pennsylvania Avenue, in the City of Philadelphia, by what is known as the Pennsylvania Avenue Subway, and also the construction of the sewers connected therewith.

General Description.

The Philadelphia and Reading Railroad enters the densely populated portion of the City of Philadelphia by means of two principal systems:

First.—That from Wayne Junction, Tioga, and Ninth St. to the Reading Terminal at Twelfth and Market Sts., formerly on the New York Division.

Second.—That from Falls of Schuylkill, Belmont and Pennsylvania Ave., also connecting with the Reading Terminal near Twelfth and Callowhill Sts., formerly on the Main Line Division.

Both of these systems are now part of the Philadelphia Division.

In 1893 both of these lines contained a very large number of grade crossings, which were becoming more dangerous as the city grew and the population on the lines of the tracks increased.

On account of the large number of tracks which formerly crossed Broad St. at grade between Callowhill St. and Pennsylvania Ave., and because Broad St. is one of the widest and finest streets in the city, being, at the point referred to, on one of the principal routes from the center of the city to Fairmount Park, special attention had been directed for some years to an arrangement between the City of Philadelphia and the Philadelphia and Reading Railroad Company by means of which this particular grade crossing could be avoided.

On December 26th, 1890, an ordinance was approved by the Mayor, giving the Philadelphia and Reading Terminal Railroad Company the right to construct the Terminal Station at Twelfth and Market Sts., and connect by means of an elevated structure with the tracks of the Philadelphia and Reading Railroad Company at:

First.—A point near Broad St. and Pennsylvania Ave., and,

Second.—A point near Ninth St. and Fairmount Ave.

In consideration of this privilege the Philadelphia and Reading Railroad Company was, by means of changes of the grades of the streets, to abolish the grade crossings at Broad St. and Lehigh Ave., at Ninth St. and Columbia Ave., and at Broad St. and Pennsylvania Ave., all without expense to the city. By this ordinance, Broad St. was to be carried over the railroad at both Pennsylvania and Lehigh Aves. The plans for this change at Broad St. and Pennsylvania Ave. provided for raising Broad St. about 21 ft., and was known as the "Hump" plan.

After the construction of the bridge, with its approaches at Lehigh Ave., in the northern section of the city, it was demonstrated clearly that a similar raising of the grade at Pennsylvania Ave., almost in the heart of the city, would not only seriously damage valuable property, but would practically ruin the finest avenue in the city, and would leave sixteen dangerous grade crossings to the westward unprovided for.

Before arrangements were made to begin work at Pennsylvania Ave., several plans were prepared by different interests, with a view of avoiding the hump on Broad St. One of these contemplated the abolishment of all grade crossings on the line of Pennsylvania Ave. and Noble St. from Thirteenth to Thirtieth Sts. by means of a tunnel extending from Hamilton St. to a point near Taney St., and the remainder of the line, including the freight yards of the railroad company, in an open subway with bridges on the line of Broad and other cross streets. This plan contemplated but a slight change in the grade of Broad St., and had the additional advantage of abolishing sixteen other dangerous grade crossings over Pennsylvania Ave., besides providing an entrance to Fairmount Park free from all danger and annoyance from passing trains.

The question of elevating the railroad tracks on the avenue was considered, but as this type of construction would result in all the intersecting streets being crossed by bridges, over which constant and heavy traffic would be passing, and particularly as, in addition to Broad St., these crossings would be over Twenty-second, Spring Garden and Green Sts., and Fairmount Ave., all principal entrances to Fairmount Park, the scheme was objected to seriously by citizens in the central and southern portions of the city. The bridge proposed at Broad St. was specially protested against as, on account of the crossing being in the center of the railroad terminal at this point, the width of the overhead bridge would have been 265 ft., thus practically making a tunnel of this length over the street, and cutting off the view on the central boulevard of the city.

After the preparation of a number of studies by the engineers of the city, of the Philadelphia and Reading Railroad, and by the late John A. Wilson, M. Am. Soc. C. E., Consulting Engineer for the Railroad Company, a plan and estimates were prepared for depressing the tracks and carrying them in subway and tunnel beneath all the streets from and including Broad St. to the west as far as Thirtieth St., permanently abolishing all the grade crossings, providing for the railroad business and for the maintenance of adequate track connections along the avenue from Thirteenth to Twenty-second Sts.

The advantages of a subway are that it does not disfigure the streets which cross it, the grades being so arranged that the superstructures of the bridges are beneath the level of the streets, and the

contour therefore remains unbroken. By means of the tunnel from Twenty-second St. to the west, the tracks are removed from sight, and what was formerly a half-blighted section of the city will be reclaimed, obtaining for the city a splendid driveway and an appropriate entrance to Fairmount Park freed from all objectionable features. It will also result in a partial financial return in increased taxes. From a railroad standpoint, it will do away with the constant expense of watchmen, the maintenance of safety gates, and will allow of the rapid running of trains on a clear permanent way, besides practically doing away with all claims arising from accidents to the public.

Legislation.

In the spring of 1894, a printed draft of an ordinance for the work generally described above, together with lithographs of bird's-eye views of Pennsylvania Ave. as it existed at that time and as it was proposed to change it, was placed in the hands of Common and Select Councils, the Legislative bodies of the city, together with an estimate of the total cost of the work, which was placed at $6 000 000. These lithographs are reproduced in Figs. 1 and 2 and show clearly the intent of the work to the average layman.

On March 15th, 1894, an ordinance was approved by the Mayor, then the Honorable Edwin S. Stuart, providing for the issuance of a loan of $6 000 000 to be used for the work. The ordinance provides that the city shall negotiate the loan, as specially provided, and the railroad company shall eventually pay back to the city half the cost of the work, providing that their share of the said cost in no event exceeds the sum of $3 000 000. The loan to be issued in twenty series of $300 000 each.

On March 17th, 1894, an ordinance was approved by the Mayor, for carrying out the work in substantial accordance with the general plans previously mentioned.

The work authorized by this ordinance involved the depression of the tracks and yards of the Philadelphia and Reading Railroad Company between Broad and Thirtieth Sts., including the alteration of the lines and grades of the tracks and yards between the north side of Noble St. and Callowhill St., and between Eleventh and Broad Sts.; the alteration of the lines and grades of the tracks of the Philadelphia

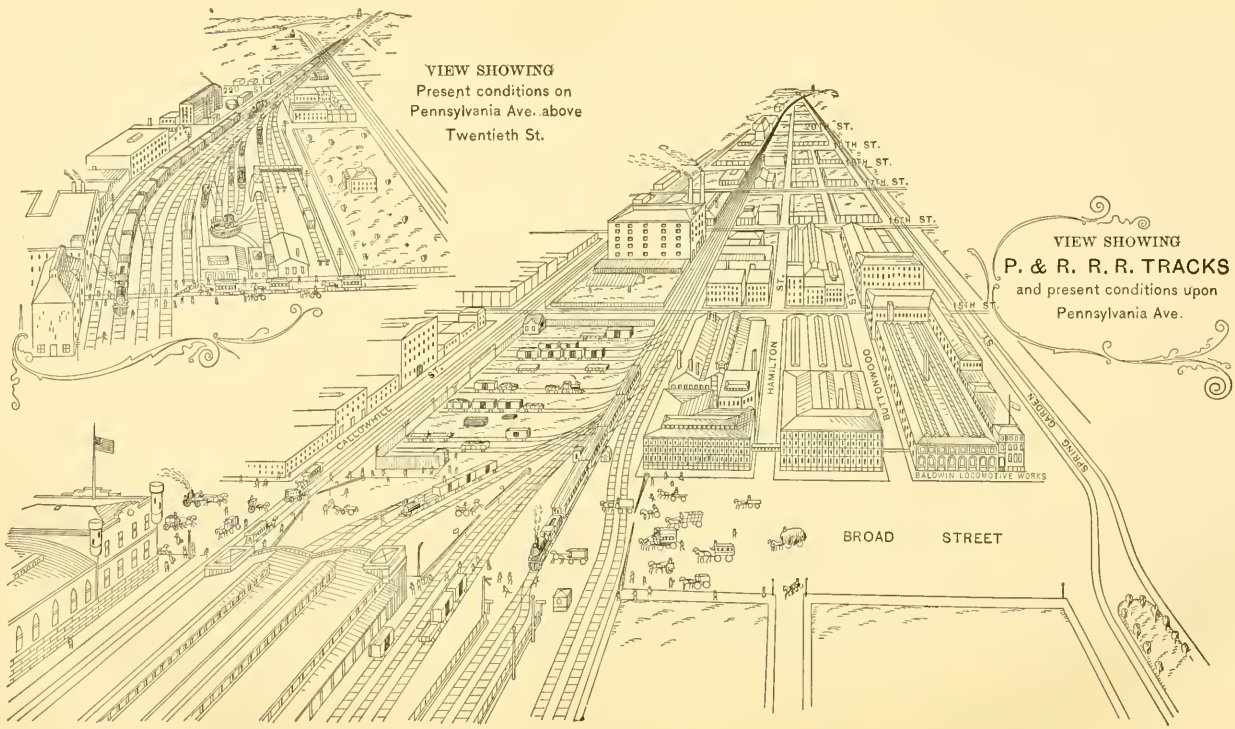

VIEW SHOWING
Present conditions on
Pennsylvania Ave. above
Twentieth St.
VIEW SHOWING
P. & R. R. R. TRACKS
and present conditions upon
Pennsylvania Ave.
20TH ST.
19TH ST.
18TH ST.
17TH ST.
16TH ST.
15TH ST.
CALOWHILL ST.
HAMILTON ST.
BUTTONWOOD ST.
SPRING GARDEN ST.
BALDWIN LOCOMOTIVE WORKS
BROAD STREET

FIG. 1.

and Reading Terminal Railroad Company, east of Broad St., and between Noble and Carlton Sts. Accompanying this depression and alteration of tracks and yards, is the alteration, construction and reconstruction of all the yard tracks, freight, engine, depot and signal buildings and other structures of the Philadelphia and Reading Railroad Company, so as to provide as much accommodation and as full and convenient a method for operation and conducting business as previously existing. Adequate track connections with the various industrial establishments along the line of the railroad were to be provided; provision was to be made for the construction of temporary tracks and bridges for maintaining travel on the railroad and upon all the intersecting streets. Bridges with adequate approaches, piers and abutments were to be built to carry Broad St., Fifteenth, Sixteenth, Seventeenth, Eighteenth, Nineteenth, Twentieth and Twenty-first Sts. over the tracks of the railroad. Bridges at Callowhill, Twelfth and Thirteenth Sts. were to be built to carry the tracks of the railroad over these streets. Provision was made for carrying the tracks in an open subway, with the necessary retaining walls and underpinning of all structures along the line, from a point near Broad St. to Hamilton St., by a tunnel from Hamilton St. to a point near Taney St., and thence by an open subway to a point near Thirtieth St. An entire reconstruction of the sewerage system between Twelfth and Thirtieth Sts. was made necessary, as well as the alteration and reconstruction of gas and water mains, electrical conduits and other municipal structures.

Twelfth, Thirteenth, Broad and a few of the other intersecting streets required some change of grade.

The lengths of the various classes of work to be constructed are as follows:

Elevated structure	959 ft.
Open subway	6 330 "
Tunnel	2 711 "
Total	10 000 ft.

On August 31st, 1894, a formal agreement* was entered into between the Receivers of the Philadelphia and Reading Railroad and the city,

*The form of agreement and ordinance in full have been filed in the Library of the Society for reference.

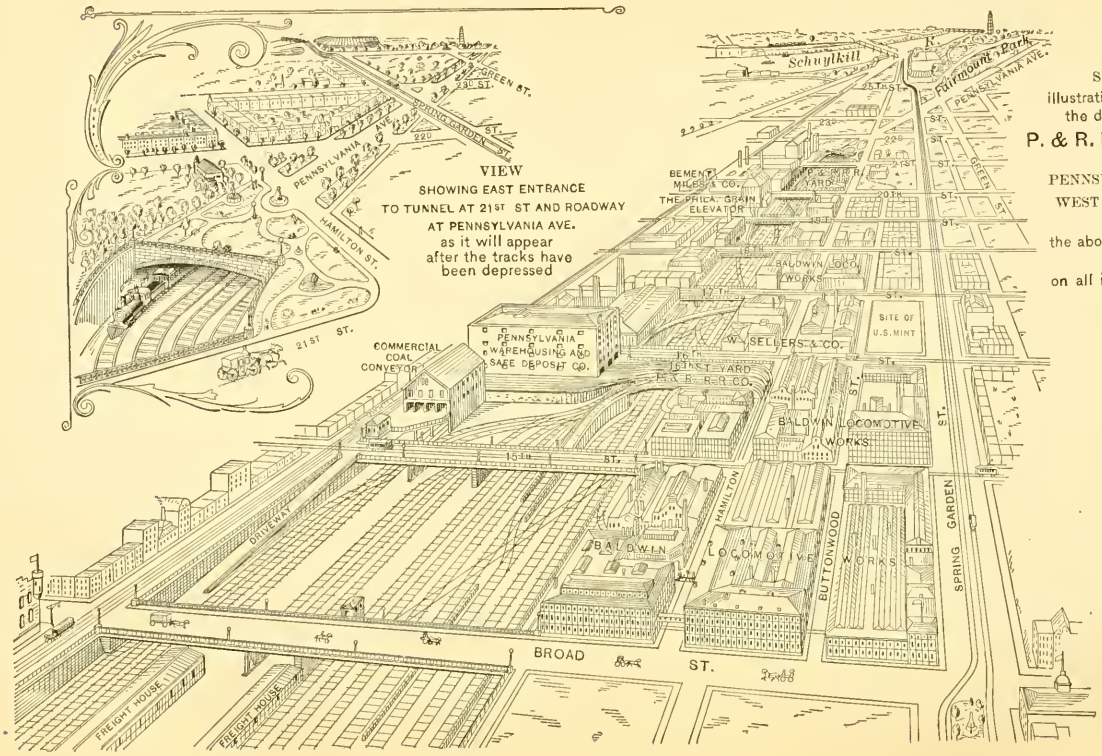

VIEW
SHOWING EAST ENTRANCE
TO TUNNEL AT 21ST ST AND ROADWAY
AT PENNSYLVANIA AVE.
as it will appear
after the tracks have
been depressed
SKETCH
illustrating the plan for
the depression of
P. & R. R.R. TRACKS
ON
PENNSYLVANIA AVE.
WEST OF BROAD ST.
and
the abolishment of Grade
Crossings
on all intersecting streets
Schuylkill
Fairmount Park
PENNSYLVANIA AVE.
GREEN ST.
SPRING GARDEN ST.
HAMILTON ST.
21ST ST.
BEMENT MILES & CO.
THE PRICA GRAIN ELEVATOR
P. & R. R.R. YARDS
BALDWIN LOCO. WORKS
SITE OF U.S. MINT
COMMERCIAL COAL CONVEYOR
PENNSYLVANIA WAREHOUSING AND SAFE DEPOSIT CO.
W. SELLERS & CO.
P. & R. R.R. CO.
BALDWIN LOCOMOTIVE WORKS
15TH ST.
ORTHWAY
HAMILTON
BUTTONWOOD
SPRING GARDEN
BALDWIN LOCOMOTIVE WORKS
BROAD ST.
FREIGHT HOUSE
FREIGHT HOUSE

FIG. 2.

in compliance with the terms of the ordinance, which provided that before the ordinance should go into effect the Railroad Company should file its acceptance of the provisions contained and a covenant should be entered into, in form to be approved by the City Solicitor, to perform all the provisions of the ordinance as far as they relate to the Railroad Company.

On the same day they also filed a bond for $500 000, as required, as security for their payment to the City of half the total cost of the work.

On August 31st, 1894, the general plans for the construction of the work were approved, as required by the ordinance, and on August 30th bids were received for the work upon the entire sewerage system, the detailed plans for which had been prepared in anticipation.

The ordinance specifically provides that all the engineering work is to be done by the City under the supervision of the Director of the Department of Public Works, and that the plans and specifications, after being prepared, are to be submitted to the officers of the Railroad Company for their approval, and copies filed with the Railroad Company and the City.

Provision is also made for the necessary revisions of the lines and grades of the streets in order to carry out the work.

Fig. 3 is a general plan of the work and the surroundings.

Soundings.

A large number of soundings, by the water-jet process, were made along the line of the subway and on the route of the sewers, in order to give the contractors for the sewers an idea of the character of the material to be excavated, and to aid in making intelligent designs for the work of underpinning the buildings and for the main work of construction of the subway proper. Subsequent excavations, adjacent to these soundings, developed the fact that they had been made carefully, and were therefore of the greatest value in designing the work and making intelligent estimates. On account of the soft nature of the rock in many places, shown on the soundings as hard rock, it would have been well if a limited number of diamond drill borings had been made in selected localities along the line, in order to determine the nature of the rock more exactly.

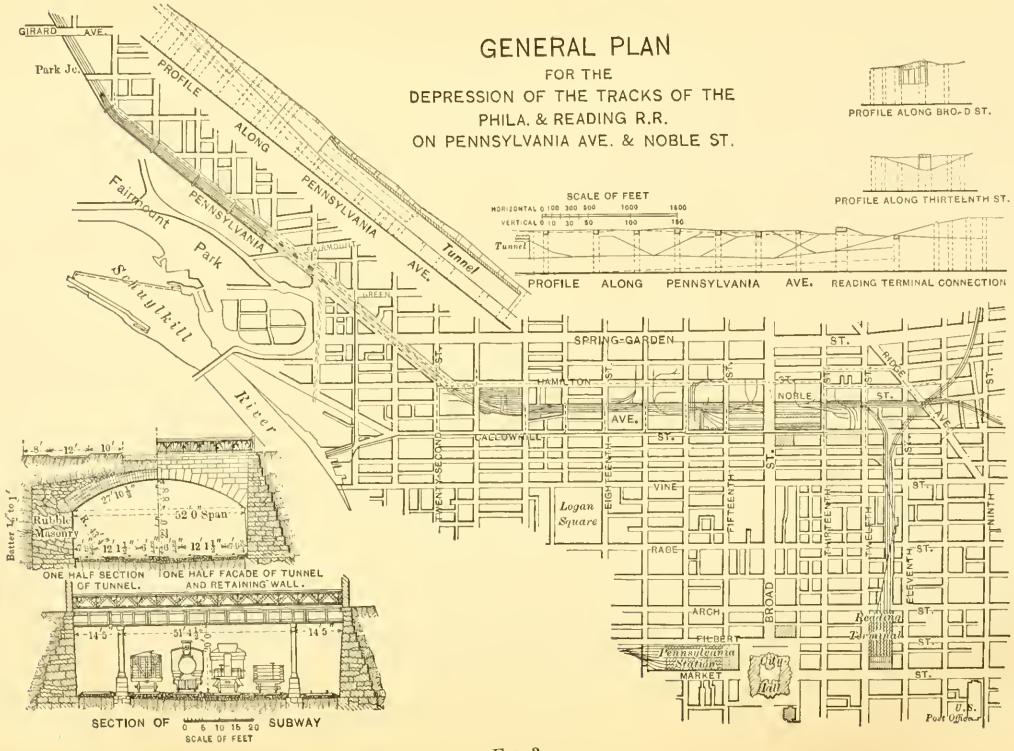

GENERAL PLAN
FOR THE
DEPRESSION OF THE TRACKS OF THE
PHILA. & READING R.R.
ON PENNSYLVANIA AVE. & NOBLE ST.
PROFILE ALONG BRO-D ST.
PROFILE ALONG THIRTEENTH ST.
SCALE OF FEET
PROFILE ALONG PENNSYLVANIA AVE. READING TERMINAL CONNECTION
GIRARD AVE.
Park Jc.
PROFILE ALONG PENNSYLVANIA AVE.
Tunnel
Fairmount Park
Schuylkill River
SPRING-GARDEN
HAMILTON
CALLOWHILL
NOBLE
VINE
RACE
ARCH
FILBERT
MARKET
Logan Square
Pennsylvania Station
City Hall
U.S. Post Office
Reading Terminal
ONE HALF SECTION OF TUNNEL.
ONE HALF FACADE OF TUNNEL AND RETAINING WALL.
Rubble Masonry
52'0" Span
SECTION OF SUBWAY
SCALE OF FEET

FIG. 3.

Sewers.

In order to drain the subway, and to provide for the old sewers cut by the excavations, three new and independent systems of main sewers were necessary to accomplish this purpose in the most satisfactory and economical manner, there being no existing systems at a sufficiently low level.

The three systems into which the sewers for carrying on this work were divided are as follows:

First. The Twenty-fourth Street and Pennsylvania Avenue System.—This system provides for the drainage west of Twenty-fourth St. on the north side of Pennsylvania Ave., intercepting all the cross sewers between Twenty-fourth and Thirtieth Sts., carrying the same beneath the tunnel at Twenty-fourth St., down Twenty-fourth St. to Powelton Ave., and on the line of Powelton Ave. to the Schuylkill River. This system was divided, for construction purposes, into two contracts.

Second. Callowhill Street System.—This system provides for the drainage of the subway and tunnel from Thirteenth to Twenty-third Sts. by means of a main sewer on Callowhill St. (which runs parallel to Pennsylvania Ave., east of Twentieth St.) with appurtenant sewers on the cross streets leading under the subway and receiving through wellholes the sewage from the old high-level sewers to the north. From Twenty-third St., the main sewer follows Powelton Ave. to Twenty-fourth St., passes along Twenty-fourth St. to Wood St. and thence to the Schuylkill River. This system is divided, for construction purposes, into three contracts.

Third. Twelfth Street System.—This system is designed to intercept the old Thirteenth St. main sewer, carry it down Buttonwood St. to Twelfth St., thence along Twelfth St. to Carlton St., where it discharges into an old main sewer on the Delaware River water-shed This system is in one contract.

Detail plans were prepared for these sewers and special specifications written upon the same lines as those in use by the Sewer Division of the Bureau of Surveys (the City Bureau having charge of all sewer work). The sewers were contracted for at a price per lineal foot for each size of completed sewer, including excavation, refilling and all appurtenances shown on the drawings, such as manholes, wellholes, spurs, etc.

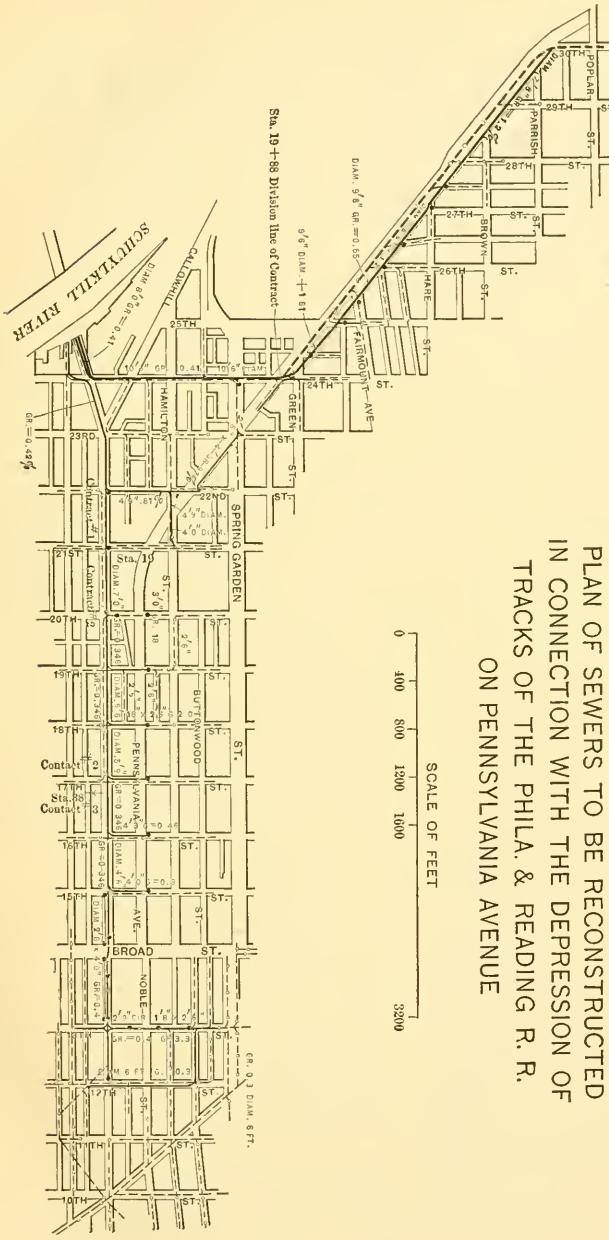

PLAN OF SEWERS TO BE RECONSTRUCTED
IN CONNECTION WITH THE DEPRESSION OF
TRACKS OF THE PHILA. & READING R. R.
ON PENNSYLVANIA AVENUE
SCALE OF FEET
0
400
800
1200
1600
3200
GIRARD AVE.
POPLAR
PARRISH
BROWN
HARE
FAIRMOUNT AVE
GREEN
HAMILTON
CALLOWHILL
SCHUYLKILL RIVER
SPRING GARDEN
BUTTONWOOD
PENNSYLVANIA
AVE.
BROAD
NOBLE
Sta. 19+88 Division line of Contract
Contract #1
Contract #2
Contract #3
Sta. 19
Sta. 38
DIAM. 9'6" GR. = 0.55
9'6" DIAM. + 1 ft
DIAM. 8'0" GR. = 0.41
GR. = 0.42%
GR. 0.3 DIAM. 6 FT.
30TH
29TH
28TH
27TH
26TH
25TH
24TH
23RD
22ND
21ST
20TH
19TH
18TH
17TH
16TH
15TH
14TH
13TH
12TH
11TH
10TH
ST.

FIG. 4.

These specifications,* while covering the whole work of construction in detail, specify some features which are rather different from usual practice, and are, therefore, worthy of special attention.

It was directed that all sewers, wherever practicable, were to be constructed in tunnel, the work being carried on day and night.

All appliances for removing excavated material were to be specially adapted for the prevention of interference with travel, and were to be subject to the approval of the Chief Engineer.

When constructed in tunnel, the space between the outside of the sewer and the roof and sides of the tunnel was to be filled compactly with rubble masonry or concrete, as directed.

On account of the light grades, the inverts on curves were to be plastered with Portland cement mortar, ½-in. thick, to reduce the friction.

In the general clauses relating to the responsibility of the contractors and to maintenance, provision is made to settle disputes between the contractors on the several sections as to the disposition of drainage by making the Director of the Department of Public Works the arbiter. The maintenance of all new work is to be cared for by the contractor or his sureties for five years from its completion. The manner, the times of carrying on the work and the force used in construction are to be subject to the approval of the Chief Engineer.

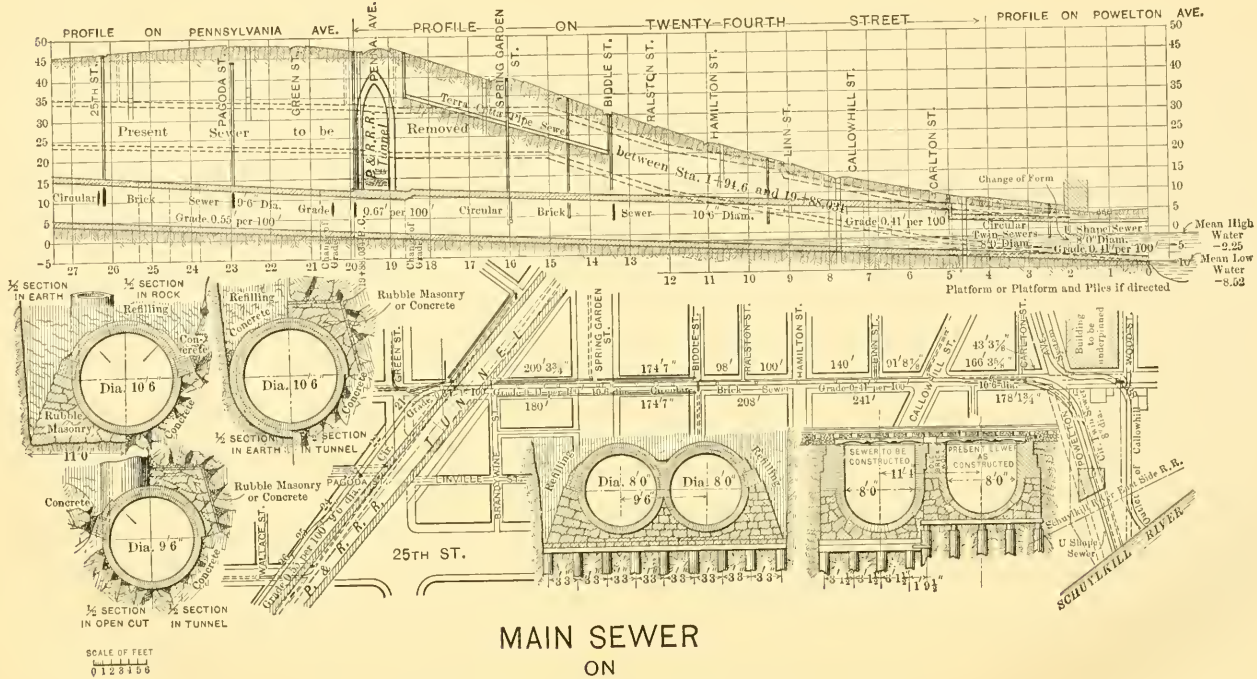
The general drainage plan of this work is shown in Fig. 4.

The time for the completion of the work was fixed by the department at four months from the date of the order to proceed.

Bids were received on August 30th, 1894, and on September 4th and 5th contracts were executed, as follows:

Contractors.	Limit of Contract.
Twenty-fourth St. and Pennsylvania Ave. System:	
Contract No. 1.—Ryan & Kelly	$102 000
" No. 2.—J. A. Mundy & Bro.	138 000
Callowhill St. System:	
Contract No. 1.—C. P. Grim & Co.	94 000
" No. 2.—Geo. W. Ruch	75 000
" No. 3.—John McCann	47 000
Twelfth St. System:	
John McCann	25 000

* The full specification has been placed on file in the Library of the Society.



MAIN SEWER
ON
24th ST. AND PENNSLVANIA AVE.,
Between the outlet at Powelton Ave., and 30th St.

FIG. 5.

The order to proceed with the work was given to all contractors alike on September 10th, 1894, and it was directed that the work be proceeded with, day and night.

Detailed descriptions of the work on each contract follow.

Twenty-fourth Street and Pennsylvania Avenue System. Contract No. 1.—Located on the line of Powelton Ave., Twenty-fourth St. and Pennsylvania Ave., from the Schuylkill River to Station 19 + 88. Ryan and Kelly, Contractors.

This contract extended from the Schuylkill River at the foot of Powelton Ave. to the north of and alongside of an old 8-ft. U-shaped sewer; passed, with very close clearance, under the tracks of the Schuylkill River East Side Railroad (Baltimore and Ohio System) and, at a distance of about 205 ft. from the outlet, changed to an 8-ft. twin sewer. At the wharf, the elevation of the inside bottom of the invert is — 7.5, Philadelphia City datum. This datum is 2.25 ft. above mean high water and 8.52 ft. above mean low water in the Delaware River at Philadelphia. It is thus seen that the inside bottom of the sewer is only 1.02 ft. above mean low water. From this point, for a distance of about 150 ft., the sewer was constructed upon piles with a yellow pine platform and masonry cradle. The details of this section of the sewer are shown in Fig. 4. At about Station 1 + 50 the sewer runs beneath an old warehouse. Here, the piles were omitted and a timber platform 22 ins. thick was substituted. As will be seen by reference to Fig. 5, the pile work was below low tide, and the excavation for a distance of over 100 ft. from the outlet was through an old wharf, making the use of a coffer dam impracticable. A diver was employed to cut off the tops of the piles after they had been driven, and the platform and cradle were placed in position at low tide, bulkheads being used wherever practicable to facilitate the work.

The U-shaped sewer was covered with steel I-beams and buckle-plates filled in on top with concrete. This construction was necessitated on account of the low level of the railroad tracks crossing at this point. It will be noticed by reference to the plans that the new main sewer on Twenty-fourth St. is 10 ft. 6 ins. in diameter. This could not be carried under the railroad tracks and preserve the required elevation of the inside bottom; hence, a separating chamber, placed at the intersection of Twenty-fourth St. and Powelton Ave., and leading to a pair of circular sewers 8 ft. in diameter, was designed. Where these

twin sewers passed under the tracks the section was again changed, as noted above. The U-shaped portion of the old sewer on this line was retained from the old warehouse to the outlet, and the new sewer was built to the north of it.

From the point where the twin sewers commence, to the end of the separating chamber, the foundation was upon two layers of 3-in. yellow pine plank laid at an angle of 60° to each other. From the separating chamber to the north side of Callowhill St. a simple masonry cradle was used, and at about this point the excavation was largely through rock. Just south of Hamilton St. tunneling was started, and continued to the end of the contract at Station 19 + 88. It was entirely through rock, with the exception of a gravel pocket beneath Pennsylvania Ave. A tunnel was also driven beneath Callowhill St. through soft material so as not to interfere with traffic on the street.

The old sewer on Twenty-fourth St. was 10 ft. in diameter, and was very old. There was a heavy and constant flow of sewage through it, and, owing to the heavy grade, the invert in many places was practically worn out and washed away. In several places, and for considerable distances, there was no brick-work whatever remaining. The new sewer was constructed on essentially the same alignment as the old one, but with a much flatter grade. Up to a point near Hamilton St., it was necessary to carry the water during construction by means of a flume of wood, rectangular in section and 6 ft. square, constructed under the west sidewalk of Twenty-fourth St., and to remove the old sewer entirely, in order to build the new one. This flume extended from Hamilton St. to about Station 1 + 90 where it discharged into the old U-shaped sewer. From Hamilton St. north, by careful working, a tunnel was driven, and the new sewer was constructed beneath the old one, which was in active service. Several breaks occurred in the bottom of the old sewer, allowing the sewage to fall into the workings beneath, but in no case causing injury or loss of life. Entrance to the tunnel was obtained through four side-shafts with drifts into the main tunnel. These shafts were generally located in the small cross streets off the line of Twenty-fourth St.

Of the total length of main sewer constructed, 1 988.03 ft., the length in tunnel was 1 001.03 ft. Ingersoll-Sergeant drills with 3½-in. cylinders were used, fed with compressed air through 3-in. and 4-in. service pipe from an Ingersoll-Sergeant Class "A" compressor with a

capacity of twelve drills. The compressors were supplied with steam from two 80 H.-P. horizontal boilers. Ten hoisting engines and twelve pumps constituted part of the contractors' plant.

The blasting for the flume, referred to, seriously damaged a number of houses on the west side of Twenty-fourth St., which were in a very bad condition previous to beginning work. This damage was made good by the contractor, under the terms of his contract.

Progress in making the excavation for the new sewer tunnel was

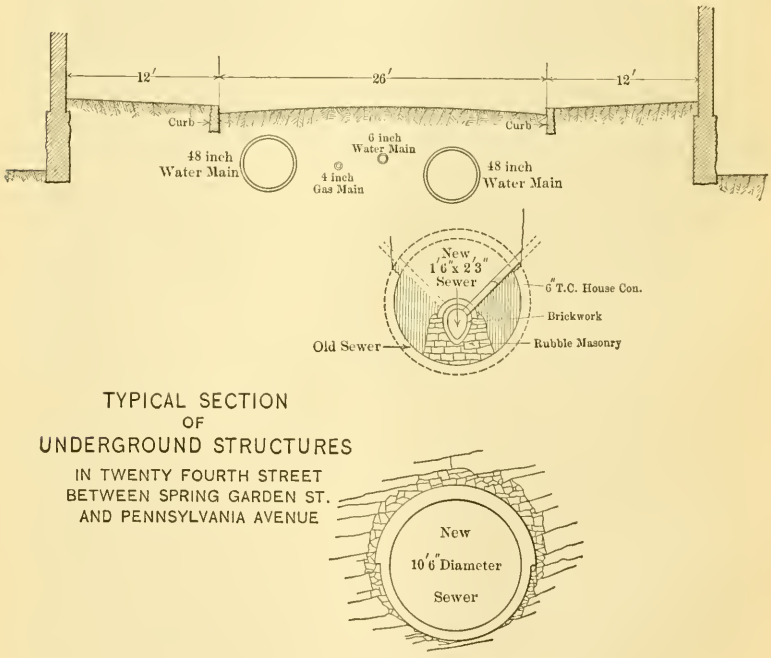


FIG. 6.

necessarily slow, on account of the dangerous proximity of buildings, the old sewer and one 6-in. and two 48-in. water mains in the street above the tunnel. Fig. 6 is a section through the street, between Spring Garden St. and Pennsylvania Ave., in which these structures were found. Work was begun September 10th, 1894, and was pushed vigorously day and night. The last sewer connection was made May 6th, 1895. The photographs on Plate I were taken from the open cut and tunnel work on this contract.

FIG. 1.—OPEN CUT ON 24TH STREET, NORTH OF CALLOWHILL STREET.

FIG. 2.—TUNNEL ON 24TH STREET, UNDER PENNSYLVANIA AVENUE.

TABLE No. 1.—Lengths, Sizes and Prices Paid.

Sizes and Shapes.	Contract price, per lineal foot.	Total length, in feet.	Length in tunnel, in feet.
8 ft. diameter, U-shaped	$47.78	205.30	.....
8 ft. diameter, twin	58.51	293.32	.....
10 ft. 6 ins. diameter with 12-in. pipe.	43.00	1 337.08	877.70
9 ft. 6 ins. diameter	45.00	123.33	123.33
Separating chamber	86.02	29.00	.....
		1 988.03	1 001.03

Total payment to contractors: $92 586.42.

Twenty-fourth Street and Pennsylvania Avenue System. Contract No. 2.—Located on the line of Pennsylvania Ave. from Station 19 + 88 to Thirtieth St. James A. Mundy and Brother, Contractors.

This is a continuation of Contract No. 1, and completes the system by making connection with the old sewer on Thirtieth St. The sewer, where it crosses under Twenty-fourth St., is 9 ft. 6 ins. in diameter and gradually decreases in size to 7 ft. 6 ins., which is the size of the old sewer on Thirtieth St. Of the total length of main sewer on this contract, 3 470.66 ft., the length in tunnel was 2 878.02 ft. Access to the tunnel was obtained through sixteen shafts, generally located where manholes or wellholes occurred. Very little soft material was encountered in making the excavation, although frequent timbering in isolated places was required, on account of the dip and seamy character of the rock, which was a mica schist of very irregular character and hardness, in some places being badly decomposed, even where 40 ft. beneath the natural surface. At the lower end of the contract, on the curve running west from Twenty-fourth St., the rock was unusually slippery when exposed to the air, and required careful watching. Just west of Twenty-eighth St., the tunnel excavation passed through a fill, formerly the bed of an old creek, and considerable difficulty was experienced in making the timbering stable.

From Brown St. to Thirtieth St. the sewer was built in open cut.

From Twenty-fourth St. to Fairmount Ave. the tunnel passed beneath a large number of dwelling houses and an ice-manufacturing plant, the latter in active operation and the former occupied. These structures were finally removed, upon the widening of Pennsylvania Ave., and the sewer is now in the bed of the street. With the exception of breaking a 48-in. water main, no serious injury was done

by the blasting. From Fairmount Ave. westward, the line of the sewer is generally distant from any buildings.

Drills of the Rand type, with 3½-in. cylinders, were used, and were fed with compressed air through 4, 6 and 8-in. service pipes from two Rand compressors with a joint capacity of thirteen drills. The compressors were supplied with steam from two 75 H.-P. horizontal boilers. Seven hoisting engines and six pumps constituted part of the contractors' plant.

Work was begun September 10th, 1894, and was pushed day and night. The last sewer connection was made April 16th, 1895. Fig. 1, Plate II, shows the work in tunnel on this contract.

TABLE No. 2.—Lengths, Sizes and Prices Paid.

Sizes.	Contract price, per lineal foot.	Total length, in feet.	Length in tunnel, in feet.
9 ft. 6 ins. diameter.....	$49.33	1 382.70	1 382.70
8 ft. 6 ins. diameter.....	35.99	870.26	870.26
7 ft. 6 ins. diameter.....	20.38	1 217.70	625.06
		3 470.66	2 878.02

Total payment to contractors: $126 496.15.

Callowhill Street System. Contract No. 1.—Located on the line of Wood St., Twenty-fourth St., Powelton Ave. and Callowhill St. from the Schuylkill River to Station 19 at Twenty-first St., with appurtenant sewers on Twenty-second St., Hamilton St. and Pennsylvania Ave., C. P. Grim and Company, Contractors.

This contract is the outlet and most difficult section of the Callowhill St. System. From a point near Twenty-fourth St. to the Schuylkill River, the sewer followed the line and approximately the grade of the old Wood St. sewer, which is about 4 ft. in diameter. Near the crossing of the Baltimore and Ohio Railroad, the new sewer cut through part of the old wharf referred to on the Twenty-fourth St. System. At the outlet, the elevation of the inside bottom of the invert is — 8.17, or only 0.35 ft. above mean low water.

From the outlet to Station 1 + 40 the sewer is 8 ft. 6 ins. in diameter with an 18-in. arch, and is constructed upon a platform placed upon piles and with a masonry cradle. The piles and platform extended to about Station 1 + 40, where a timber platform was

FIG. 1.—TUNNEL ON PENNSYLVANIA AVENUE, WEST OF 26TH STREET.

FIG. 2.—SEWER IN TUNNEL ON CALLOWHILL STREET

commenced. During the construction of the new sewer on the line of Wood St. it was necessary to flume the sewage from the old sewer, and, at the outlet, to work only at the lowest stages of the tide. It was impossible to place bulkheads at any point to prevent ingress of the tide, on account of the structure of the old wharf.

At the southeast corner of Twenty-fourth St. and Powelton Ave. the sewer approaches very closely to the three-story brick building of Bement, Miles and Company, used as a store house for patterns of machine tools, and, on account of the admittedly bad character of the foundation under the walls of this building, it was not considered advisable to proceed with the excavation without first underpinning the building. A supplementary contract was therefore made with the contractors for this underpinning.

The special specifications prepared for this work required the excavation to be made by sinking shafts, not adjacent to each other, and shoring the sides carefully where necessary. All loose or imperfect stones in the bottom of the old foundation were required to be removed, until such portions of the same were reached as were compact and the stones of large size. At the center of each section (generally 4 ft. long), and under the prepared bottom of the foundation wall, a timber of long-leaf yellow pine 10 by 10 ins. in section was to be placed and firmly wedged up with iron or steel wedges. The space around the timber and under the wall was filled with masonry of stone with specially flat beds (the stone from Conshohocken, Montgomery County, Pa., was specified), laid in Portland cement mortar, mixed in the proportion of 1 part cement to 1 part sand. This masonry was required to be wedged up firmly so that no space was left for settlement.

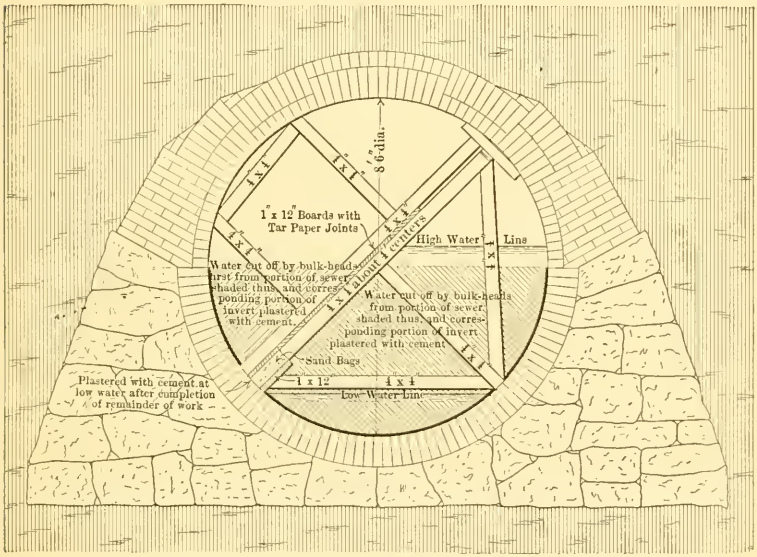
The excavation was carried down through river mud and sand for a distance of 20 ft., and was stopped on a bed of compact gravel below the bottom of the excavation required for the sewer. A bed of concrete about 4 ft. wide and 3 ft. deep was laid as a footing course. The building thus underpinned contained a number of cracks previous to the operation, but no new cracks were started, nor were any of the old ones increased in size. The price paid for the underpinning, including the excavation and all appurtenances, was $18 per cubic yard.

On the line of Powelton Ave., the excavation was in open cut through a bed of gravel as far as Callowhill St., where a tunnel was

driven beneath this street and Twenty-third St. in order to keep the streets open to travel. While driving this tunnel, and on the line of Twenty-third St., an old sewer, about 3 ft. in diameter, was encountered, of which there was no record on the city drainage plans. This sewer carried very large volumes of water at certain hours of the day when the dye mills in the vicinity emptied their tanks. This caused the contractors much annoyance and expense in maintaining the flow of the sewage. From Twenty-third St. to a point west of Twenty-second St., the excavation was in open cut, the contractors using, on this section, a Carson sewer-trench machine. A second unrecorded sewer, running diagonally beneath the adjacent houses, was cut about the middle of this block. The volume of water carried by it, however, was very small and was easily cared for. From a point just west of Twenty-second St. to Station 19, the end of the contract on Callowhill St., and to the end of the appurtenant sewers at Twenty-first and Hamilton Sts. and Twenty-second St. and Pennsylvania Ave., the excavation was made in tunnel, with the exception of a short length at Twenty-first and Hamilton Sts. The tunnel was entered by means of eight shafts so located as to be off the main streets. The total length of the sewers on this contract was 3 094.61 ft., of which the length constructed in tunnel was 1 751.87 ft. Under Callowhill St. the tunnel was in soft ground, the remainder was through rock of varying degrees of hardness. Drills of the Ingersoll-Sergeant, Rand and Kerner types, with 3-in. cylinders, were used. They were fed with compressed air through a 4-in. service pipe from a Blake duplex compressor with a capacity of eight drills. The compressor was supplied with steam from two vertical boilers with a combined horse-power of 70. The contractors had in use four hoisting engines and ten pumps.

The contractors on this section did not prosecute their work as rapidly as they should, and, as they finished their brick-work, they failed to plaster the inverts on curves as required by the specifications. As a consequence, they were compelled to care for the full flow of the sewers on the contracts above them, and plaster the inverts and maintain the flow of the sewer at the same time. In order to accomplish this, a longitudinal flume, the full length of the curve, was constructed, as shown in section in Fig. 7. Bulkheads of sand bags were then built at each end of the flume in the spaces shown by the smallest shaded

area, and the flow of sewage or the returning tide made to pass beneath. The smaller segment of the invert, being free from water, was then plastered. As soon as the plaster had set, the bulkheads were removed and placed in the spaces shown by the largest shaded area, and the flow made to pass on top of the flume. The larger segment was then plastered. After the flume was removed from the sewer, the spaces on the invert occupied by the framing were plastered at low water, they being purposely arranged so as to be above water at low tide and at ordinary stages of the flow in the sewer. During its



SECTION OF CALLOWHILL ST. SEWER
SHOWING FLUME USED FOR PLASTERING INVERT ON CURVES

SCALE OF FEET
0 1 2 3 4 5 6

FIG. 7.

use the flume was endangered by two severe rain storms. Sufficient warning was given, however, to enable men to enter the sewer and remove the bulkheads by throwing the sand bags on the bottom, thus reducing the obstruction to a minimum. No damage resulted in either case.

The plastering upon brick over which sewage had been running was a piece of work which required great care and skill, in order to make a good finish. In some cases, where the ground-water was

large in amount, weep holes were made through the brick. At such places, the bricks were thoroughly soaked with water, and there was considerable trouble in making the plaster adhere. In some cases, the grease from the sewage interfered with the proper adhesion, and had to be burnt off with plumber's blow-pipes. Experience with the conditions soon resulted in a very satisfactory piece of work. A quick-setting neat cement, worked as dry as possible, being used where the amount of water in the brick was greatest.

Work was begun September 10th, 1894, was pushed vigorously for a few months, and then carried on in a dilatory manner. The last sewer connection was made in November, 1895, and the plastering of the inverts was finished February 26th, 1896.

TABLE No. 3.—Lengths, Sizes and Prices Paid.

Sizes and shapes.	Contract price, per lineal foot.	Total length, in feet.	Length in tunnel, in feet.
8 ft. 6 ins. diameter, 18-in. arch.....	$72.50	140.00	.....
" " " 13½-in. arch.....	26.75	1 244.24	196.00
7 ft. diameter.....	27.15	516.73	516.73
4 ft. 9 ins. diameter.....	20.00	692.64	692.64
4 ft. diameter.....	17.25	250.00	95.50
3 ft. diameter.....	20.50	251.00	251.00
4 ft. by 2 ft. 8 ins., egg-shaped.....	8.00	not built.	.....
		3 094.61	1 751.87

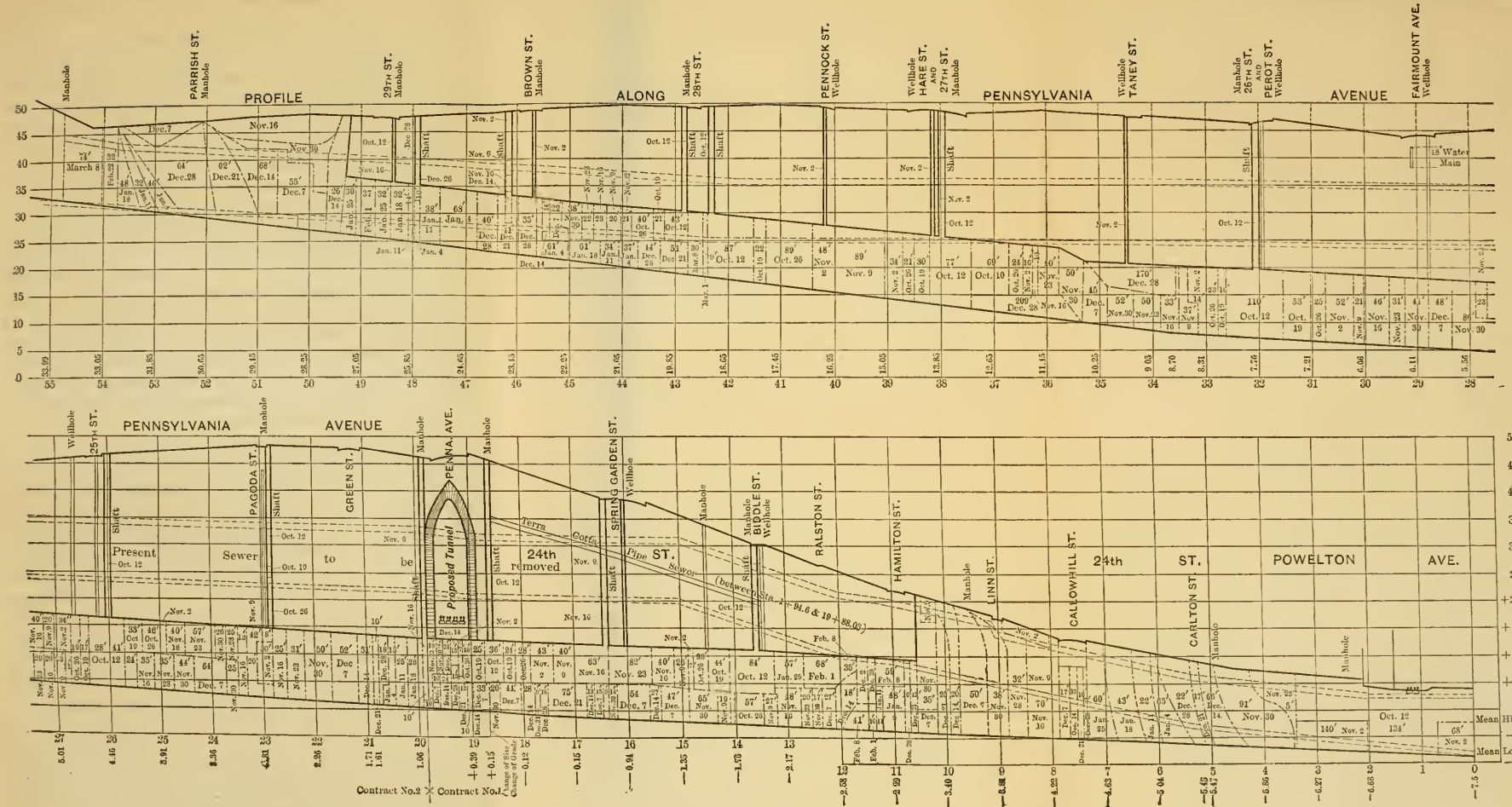
Total payment to contractors: $80 342.75.

The 4 ft. by 2 ft. 8 in. egg-shaped sewer was not constructed, on account of its close proximity to the north wall of the tunnel, which had to be excavated in rock immediately adjacent. It was omitted, therefore, in this contract, and constructed after the excavation of the tunnel wall was made.

Callowhill Street System. Contract No. 2.—Located on the line of Callowhill St., from Station 19 at Twenty-first St., to Station 38 at Seventeenth St., with appurtenant sewers on Seventeenth, Eighteenth, Nineteenth and Twentieth Sts., George W. Ruch, Contractor. This was a continuation of Contract No. 1, and was the middle contract of the system.

The whole of the 2 953.20 ft. of sewers constructed on this contract was built in tunnel, access to the headings being obtained through twelve shafts. At the western end of the line, the tunnel was driven through rock, and very little or no timbering was required. From

PLATE III.
TRANS. AM. SOC. CIV. ENGS.
VOL. XLIV, No. 878.
WEBSTER AND WAGNER ON SUBWAY SEWER CONSTRUCTION.



Station 19 to a point near Nineteenth St., the excavation was through rock which decreased in hardness as the work progressed toward the east. Rock was encountered on Twentieth St., that under Pennsylvania Ave. being specially hard. On the other hand, the sewer on Seventeenth St. was driven through a micaceous sand, very slippery and dangerous to handle, and requiring close sheathing in the headings. The most serious accident on the work occurred on the Callowhill St. sewer, where it passed under the old high-level sewer on Nineteenth St. A cave-in occurred at this point, caused by insufficient timbering, the sewage from above filling the entire workings below. There was also trouble on Callowhill St., between Twentieth and Twenty-first Sts., caused by blasting. The blasts were so severe as to affect the adjustment of machine tools in the shops of Messrs. Bement, Miles and Company, immediately adjoining the line of the sewer, and spoiling some work which was on the machines. After several cautionary orders, the Department was compelled to direct the detailed operations of firing the charges, and, as a result, the contractor, at the termination of the work, sued the city for damages resulting from the issuance of such an order. The suit was entered by the contractor in the sum of $57 377.46, afterward revised, as agreed upon by counsel, to $52 785.16. The suit was tried in November, 1897, and was interesting on account of the testimony of an expert nature, presented by the contractor to show that the Chief Engineer had exceeded his authority by issuing orders directing the manner in which the blasting was to be done, when no special method was specified. The court ruled that the Chief Engineer had the right to issue such instructions if they were reasonable.

Testimony was presented by the city to show that the progress made in driving the tunnels was as great after issuing the order as it had been before. As this testimony could not be refuted, the case was settled, and a verdict of $8 705.13 was awarded the contractor. This verdict meant a practical victory for the city, as it represented retained moneys and interest on the same held by the city, pending the settlement of the case.

Drills of the Ingersoll-Sergeant type with $3\frac{1}{2}$-in. cylinders were used. The compressed air was supplied through 3, 4 and 5-in. service pipes from a Norwalk single compressor, with a capacity of twelve drills. The compressor was supplied with steam from two

65 H.-P. vertical boilers. In addition to this machinery the contractor had in use five hoisting engines and five pumps.

Work was begun on September 20th, 1894, and the last sewer connection was made June 3d, 1895. Fig. 2, Plate II, shows the sewer construction in tunnel on Callowhill St., between Twentieth and Twenty-first Sts.

TABLE No. 4.—Lengths, Sizes and Prices Paid.

Sizes and shapes.	Contract price, per lineal foot.	Total length, in feet.	Length in tunnel, in feet.
7 ft. diameter	$29.65	967.00	967.00
6 ft. 6 ins. diameter	29.54	478.00	478.00
5 ft. 9 ins. diameter	26.29	455.00	455.00
3 ft. diameter	16.27	257.45	257.45
2 ft. 9 ins. diameter	16.27	265.25	265.25
3 ft. 3 ins. by 2 ft. 2 ins., egg-shaped	9.77	265.25	265.25
2 ft. 6 ins. by 1 ft. 8 ins., egg-shaped	10.35	265.25	265.25
		2 953.20	2 953.20

Total payment to contractors: $69 662.71.

Callowhill Street System. Contract No. 3.—Located on the line of Callowhill St., from Station 38 at Seventeenth St., to Thirteenth St., and on Thirteenth St. from Carlton St. to Buttonwood St., with appurtenant sewers on Fifteenth and Sixteenth Sts. John McCann, Contractor.

The total length of sewers on this contract was 2 467.97 ft., of which the length constructed in tunnel was 2 060.97 ft. Access was obtained through nine shafts. A considerable portion of the sewer on Sixteenth St., and the main sewer on Callowhill St. from east of Sixteenth St. to Station 38, was in rock; the remainder of the work was through soft material, much of it being a slippery micaceous sand, comparatively free from water. While the department would have allowed open cutting east of Broad St., on account of the soft character of the material, and because the depth of the sewer was not very great, the contractor elected to build the sewer in tunnel, and made very satisfactory progress, and without accident.

On account of the grade of Thirteenth St. being changed materially by the plans of the subway, it was deemed inadvisable by the department to dig up the street and construct the sewer at the time the rest of the sewer work was being done, on account of the double incon-

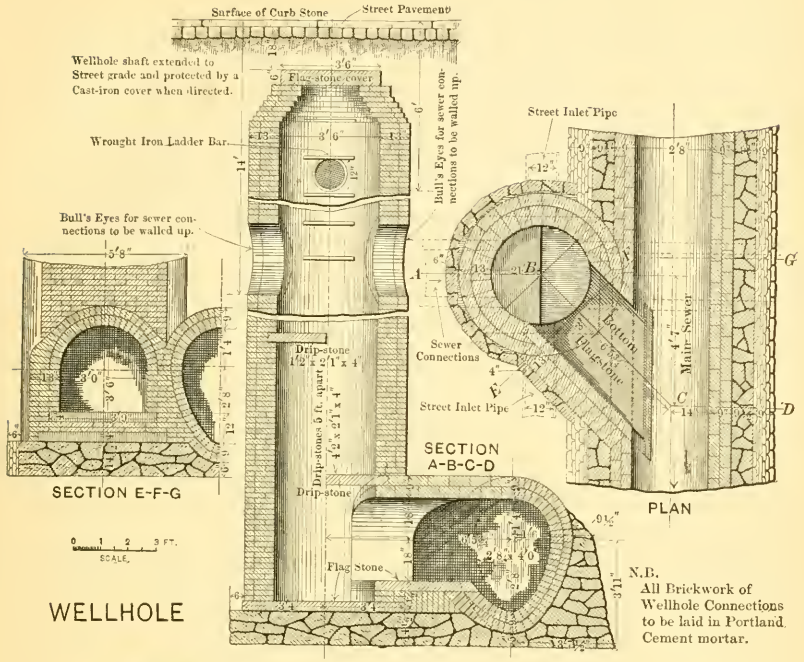
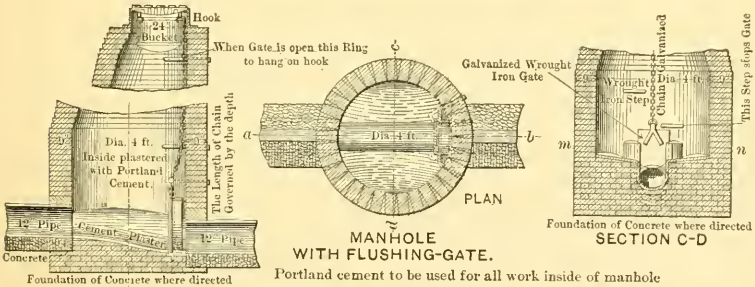
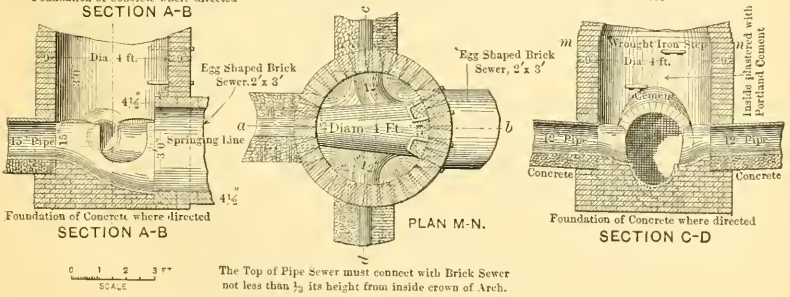


FIG. 8.





GENERAL DETAILS FOR PIPE SEWERS.

FIG. 9.

venience to the public. No work was done on Thirteenth St., therefore, under this contract.

The contractor had no power drills or compressors on the work. All the drilling was done by hand.

Work was begun September 10th, 1894, and the last sewer connection was made May 18th, 1895. This connection was delayed, on account of the failure of the contractor on Contract No. 2 to complete the tunnel at the upper end of his section.

TABLE No. 5.—Lengths, Sizes and Prices Paid.

Sizes and Shapes.	Contract price, per lineal foot.	Total length, in feet.	Length in tunnel, in feet.
5 ft. 9 ins. diameter	$27.90	390.52	390.52
4 ft. 6 ins. diameter	12.84	438.21	438.21
4 ft by 2 ft. 8 ins., egg-shaped	10.50	1 056.92	651.92
2 ft. 9 ins. diameter	11.00	not built.	not built.
2 ft. 6 ins. diameter	7.75	38.50	38.50
2 ft. 6 ins. by 1 ft. 8 ins., egg-shaped	5.75	not built,	not built.
12-in. T. C. pipe	2.40	" "	" "
4 ft. 3 ins. diameter	11.00	273.72	273.72
4 ft. diameter	11.70	253.75	253.75
Junction chamber	104.50	14.35	14.35
		2 467.97	2 060.97

Total payment to contractor: $35 976.81.

Twelfth Street System.—This contract extends on Twelfth St., Buttonwood St. and Thirteenth St., from Carlton St. to Whitehall St. John McCann, Contractor.

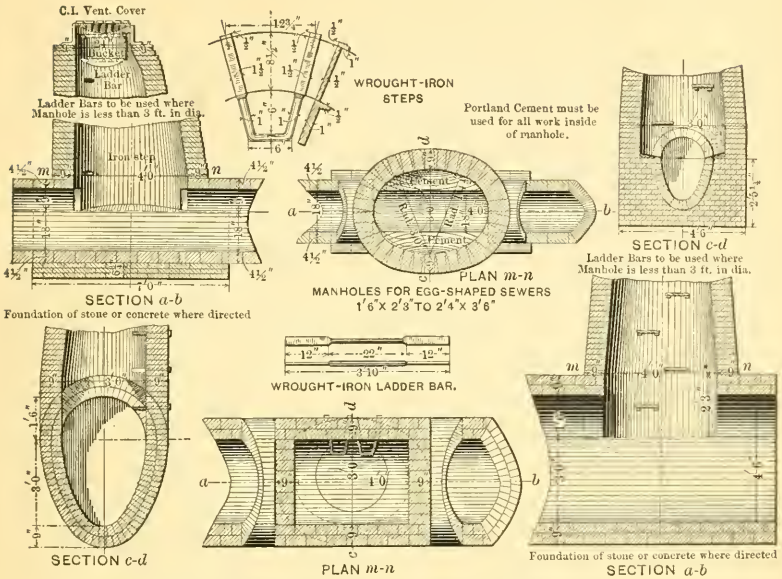
The sewer on this system was near the surface of the street and was constructed entirely in open cut. There were no points of special interest in connection with it.

Work was started September 10th, 1894, and the connection to the old sewer was made on December 29th, 1894.

TABLE No. 6.—Length, Size and Price Paid.

Size.	Contract price, per lineal foot.	Length.
6 ft. diameter.	$16.00	1 398.00

Total payment to contractor: $22 823.49.

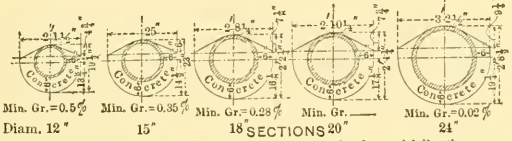


MANHOLES FOR EGG-SHAPED SEWERS OF MEDIUM SIZE.

GENERAL DETAILS FOR EGG-SHAPED SEWERS.

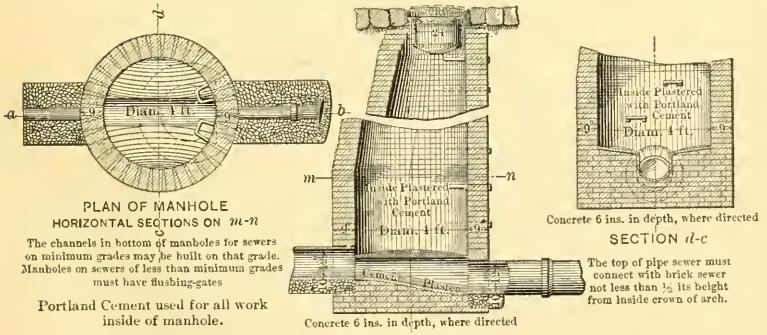
SCALE

FIG. 10.



Sewers must not have less than minimum grades, unless by special directions.

House connections not less than 15 ft. apart on each side by means of single Y's



GENERAL DETAILS FOR PIPE SEWERS.

SCALE

FIG. 11.

TABLE No. 7.—Prices Fixed in All Sewer Contracts.

Additional rubble masonry in place, per cubic yard, including all appurtenances	$4.50
Additional brick masonry, in place, per cubic yard	9.50
Additional manhole with cover, without bucket	25.00
Additional wellhole, per vertical foot	5.00
Each inlet with cover or grating, as follows:	
No. 1	96.00
No. 2	87.00
No. 3	81.00
No. 4	45.00
No. 5	40.00
Additional terra-cotta pipe not in concrete, per lineal foot	1.10
" " " " " " "	1.35
" " " " " " "	0.50

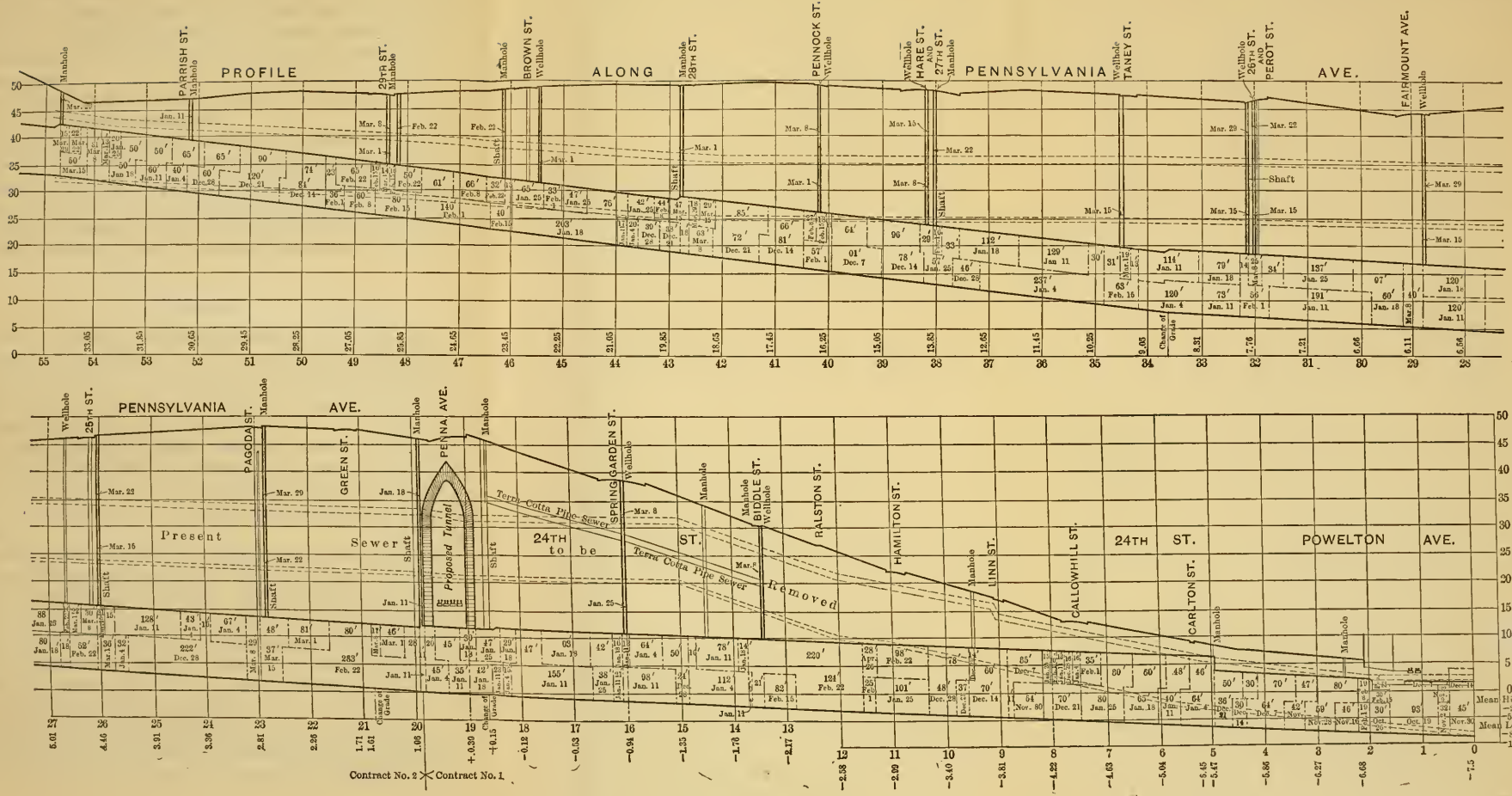
TABLE No. 8.—Summary of Sewer Work.

System.	Contract.	Shafts.	Total length in feet.	Length in tunnel, in feet.	Total cost.
Twenty-fourth St. and Pennsylvania Ave.	No. 1	9	1 988.03	1 001.03	$92 586.42
Twenty-fourth St. and Pennsylvania Ave.	No. 2	16	3 470.66	2 878.02	126 496.15
Callowhill Street	No. 1	8	3 094.61	1 751.87	80 342.75
Callowhill Street	No. 2	12	2 953.20	2 953.20	69 662.71
Callowhill Street	No. 3	9	2 467.97	2 060.97	35 976.81
Twelfth Street	.....	....	1 398.00	.....	22 823.49
		54	15 372.47	10 645.09	$427 888.33

General Sewer Notes.

Except on Twelfth St., Buttonwood St. and Callowhill St., west of Twenty-second St., no provision was made in the sewers for house connections, on account of the depth of the main sewer beneath the street. Where it was necessary to provide for such connections, auxiliary pipe sewers were constructed at a higher elevation, and these emptied into the deep sewers through wellholes at suitable intervals.

In many cases, where high-level sewers existed already, it was possible to maintain them with their house connections intact by building


PROFILE ALONG PENNSYLVANIA AVE.
PARRISH ST.
29TH ST.
BROWN ST.
28TH ST.
PENNOCK ST.
HARE ST. AND 27TH ST.
TANEY ST.
26TH ST. AND PEROT ST.
FAIRMOUNT AVE.
Manhole
Wellhole
Shaft
Change of Grade
PENNSYLVANIA AVE.
25TH ST.
PAGODA ST.
GREEN ST.
PENNA. AVE.
SPRING GARDEN ST.
BIDDLE ST.
RALSTON ST.
HAMILTON ST.
LINN ST.
CALLOWHILL ST.
24TH ST.
CARLTON ST.
POWELTON AVE.
Present Sewer
Proposed Tunnel
Terra Cotta Pipe Sewer
24TH ST. to be removed
Contract No. 2
Contract No. 1
Mean High Water −2.25
Mean Low Water −8.62

the new sewer in tunnel and not disturbing the surface of the street. Where it was necessary to give both old and new sewers the same alignment, side shafts were sunk, or the old sewer was carried across the shaft in a flume during construction. Wherever practicable, the shafts were sunk where manholes were required, thus accomplishing a double purpose.

The details of manholes, wellholes, etc., are shown in Figs. 8 to 11, which explain themselves sufficiently.

Plate III shows the weekly progress made in excavation in tunnel and open cut on the Twenty-fourth St. and Pennsylvania Ave. System, and Plate IV shows the weekly progress made in laying the brickwork on the same system. As this work was pushed rapidly, the data may prove of interest. Work was always carried on in the tunnel excavation practically at night. Brickwork was only laid by day.

Engineering.

Under the ordinance authorizing this work, the Director of the Department of Public Works, who has supervision of the Engineering Bureaus of the City, was authorized and directed to appoint such assistant engineers, draughtsmen and inspectors as might be necessary for the efficient execution of the works.

The work was placed in charge of George S. Webster, M. Am. Soc. C. E., Chief Engineer, Bureau of Surveys, which Bureau has jurisdiction over such work, under the Department of Public Works. Mr. George E. Datesman is the Principal Assistant Engineer of this Bureau.

In June, 1894, Samuel Tobias Wagner, M. Am. Soc. C. E., was appointed First Assistant Engineer in charge of the work, and Mr. Charles H. Swan, Chief Draughtsman. Mr. Swan had for some time previously been engaged upon the preparation of the plans. In August, 1894, Mr. Richard I. D. Ashbridge was appointed Second Assistant Engineer in charge of the work of construction of the sewers. Mr. Charles H. Ott, with a corps of assistants, was delegated to assist in giving lines and grades.

Owing to the difficulties of giving the lines and grades on this work, and the rapidity with which it was carried on at a large number of points at the same time, and on account of the uniformly good

results obtained, the greatest credit is due to the engineers in the field.

Under the ordinance, the Director of the Department of Public Works was directed to confer during the progress of the work with an engineer to be appointed by the Philadelphia and Reading Railroad Company, in regard to carrying out the specifications and securing the proper performance of the contracts. On March 30th, 1894, the late John A. Wilson, M. Am. Soc. C. E., was appointed Consulting Engineer to represent the Railroad Company. After Mr. Wilson's death, which occurred on January 19th, 1896, Joseph M. Wilson, M. Am. Soc. C. E., was appointed Consulting Engineer. Mr. John A. Wilson was closely identified with the preparation of the studies and the details of the work, with special reference to the railroad interests.

DISCUSSION.

Messrs. Webster and Wagner.

GEORGE S. WEBSTER and SAMUEL TOBIAS WAGNER, Members, Am. Soc. C. E. (by letter).—It will be noticed by an examination of Fig. 4, which is the main drainage plan of the sewers, that the grades on the main lines are very light for the conditions which had to be met. In place of compensating for the velocity on the curves by increasing the grades, it was deemed expedient to keep the same rate of grade throughout and add to the velocity by reducing the friction. This was done by plastering the inverts on curves with Portland cement mortar ½ in. thick. This plastering was usually done after the invert was laid and before the centers for the arch were placed. In the case of Contract No. 1 of the Callowhill Street System the work of plastering was delayed until the sewage was passing through the system, with the result already referred to in the paper.

All the brickwork of the sewers, except that in the wellholes, was laid in natural cement mortar. In the wellholes, Portland cement was specified. In all cases the mortar was mixed in the proportion of 1 part of cement to 2 parts of sand. Briquettes, 1 sq. in. in cross-section, when made from natural cement mortar from the mixing box, were required to develop an ultimate tensile strength of 40 lbs., after having been 1 day in the air and 6 days in water. In the case of Portland cement, under the same conditions, a strength of 150 lbs. was required.

The other requirements of the natural cement, which, of course, represented the greater part of the work, were as follows: It shall weigh not less than 112 lbs. per imperial bushel. The residue upon a No. 50 sieve shall not exceed 4% by weight, upon a No. 100 sieve 25%, and upon a No. 200 sieve 50 per cent. Pats of cement ½ in. thick, temperature between 60 and 70° Fahr., shall develop initial set in not less than 10 minutes and hard set in not less than 30 minutes, the amount of water used being just sufficient to form a stiff plastic paste.

The following tensile strengths were required:

24 hours (in water after hard set)	75 lbs.
7 days (1 day in air, 6 days in water)	150 "
28 days (1 day in air, 27 days in water)	250 "

Mortar composed of 1 part of cement and 1 part of standard quartz sand was required to develop an ultimate tensile strength of 75 lbs. after 7 days.

All concrete used was composed of 1 part of natural cement, 2 parts of sand, and 4 parts of stone or furnace slag.

The results of all soundings made adjacent to the line of the sewer were platted on the working drawings of the sewers, for the benefit of the contractors in making their estimates. The following note was placed upon all such plans: "Soundings shown are not guaranteed or

Messrs. Webster and Wagner.

binding upon the City of Philadelphia." As a result of this note no claims were made where any differences between the soundings and the workings were found to exist.

A summary of the character of the material encountered is approximately as follows: Beginning at Thirteenth St., where the depths of the sewers were smallest, the excavation was through clay and gravel. The invert of the sewer, however, usually rested upon rotten rock of a micaceous nature. The cross sewer on Fifteenth St. was mostly in gravel, and the workings naturally contained a considerable amount of ground-water. On Sixteenth St. a considerable amount of rather hard micaceous rock was met, the invert being entirely in this material. From this point to between Nineteenth and Twentieth Sts. clay, gravel and rock of varying degrees of hardness, irregularly located, was encountered. On Seventeenth and Eighteenth Sts. very rotten micaceous rock, acting like quicksand, was discovered, requiring great care in the workings. Between Nineteenth and Twenty-second Sts., and on the Twentieth St. sewer, hard gneiss was found. As the work approached the river the rock gradually ran out, until, between Twenty-third and Twenty-fourth Sts., it had entirely disappeared.

On Twenty-fourth St. from Callowhill St. to Pennsylvania Ave. reasonably hard, and in some cases very hard, gneiss was met. Beneath Pennsylvania Ave. was a bed of gravel, afterward discovered to be of considerable extent; the sewer invert, however, was on rock. From this point to Twenty-ninth St. the excavation was through soft micaceous rock which blew to pieces and came out of the shafts to all purposes in the form of sand. From Twenty-ninth to Thirtieth St. the excavation was through clay and gravel, as the sewer was not deep.

The character of the rock east of Twenty-ninth St. was the same 40 ft. below the surface as immediately under the top soil.

The difficulty of making deep open-cut excavations adjacent to buildings, without causing settlement, is well known. This is especially true if the foundations of the buildings be inadequate, or be upon a formation which is readily affected by the action of the elements upon exposure, or if the buildings be old and poorly constructed. In many cases the trouble is aggravated if it is necessary to use an explosive to assist in the removal of the excavated material.

If, on the other hand, the excavation is made in tunnel, with careful timbering in soft material, or with properly regulated charges when blasting through rock, the danger is materially lessened, especially if all open spaces between the structure which is to be built and the surrounding material are compactly filled with masonry, carefully laid.

The construction of these sewers has been an excellent proof of the truth of this statement. Probably seven-eighths of the entire line was through streets lined on either side with buildings of all kinds, dwellings, store-houses and manufactories. In no case, where the sewers

were constructed in tunnel, was any damage done to property, with the exception of the cases referred to in the paper. All such damage was caused by blasting, and, without doubt, was brought about by the use of unduly large charges. In the case mentioned in Contract No. 2, Pennsylvania Avenue System, a 48-in. water main, immediately over the tunnel, was broken by blasting, and an ice manufacturing plant, similarly situated, was compelled to stop work on account of the leakage of the joints in the ammonia pipes, which, it was claimed, was caused by the blasting. In this latter case it was the intention to remove the plant by widening the street, so that the damage was not unexpected. On Contract No. 2, Callowhill Street System, the only damage was caused by the interference with the adjustments of very sensitive machine tools in an adjoining plant. In this case, further damage was prevented by the Chief Engineer, who gave instructions regulating the size of the blasts. The damage in this case, recovered in court from the contractor, amounted to $300, which was the full amount claimed by the plaintiff. In no case, where the sewers were in tunnel, was there any damage whatever caused by settlement, nor were any settlements in the street paving afterward discovered, except in a few cases over the points where shafts had been sunk in order to reach the tunnel. Messrs. Webster and Wagner.

The only open-cut work was in comparatively shallow depths. On Callowhill St., between Twenty-second and Twenty-third Sts., there was a slight settlement of the curb and part of the sidewalk pavement nearest to the trench. This was rectified by the contractor at a slight expense.

On Twenty-fourth St. considerable damage to adjoining buildings was caused by the construction of the temporary flume, because it was excavated through rock, and, also, because of the very old and dilapidated character of the buildings. The expense of repairing these buildings was borne by the contractor, as already stated. The location of the flume on the sidewalk was determined by the contractors as an incidental to the work.

The effect of the sub-soil drainage upon the surrounding neighborhood, caused by the construction of the sewers, has been marked. A very considerable amount of this water was encountered on the cross streets from Fifteenth to Eighteenth Sts., and was tapped into the sewers. When the excavation for the retaining walls was made, afterward, no trouble whatever was caused by water, in many cases the soil being perfectly dry where formerly running sand was encountered.

It is the purpose of the writers to present in a future paper a description of the construction of the Subway and Tunnel proper, in which the work of underpinning, the construction of the retaining walls and the bridges, as well as the special railroad and municipal features of the operation, will be elaborated.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

No. 879.

EXPERIMENTS ON THE FLOW OF WATER IN THE SIX-FOOT STEEL AND WOOD PIPE LINE OF THE PIONEER ELECTRIC POWER COMPANY, AT OGDEN, UTAH. SECOND SERIES.

By CHARLES D. MARX, M. Am. Soc. C. E., CHARLES B. WING, Assoc. M. Am. Soc. C. E., and LEANDER M. HOSKINS, C. E.

PRESENTED MARCH 7TH, 1900.

WITH DISCUSSION.

I.—OBJECT AND METHODS.

General Plan.

The following experiments made in June and July, 1899, are supplementary to those made by the writers in August, 1897, and described in a previous paper.* The main object was the same as before; to determine the relation between the mean velocity of flow in the pipe and the loss of head between certain definite points. The methods used, being in general the same as in the former work, need not be described fully here; it will suffice to indicate the points of difference in the apparatus used, and in the methods of making and reducing the observations.

* *Transactions*, Am. Soc. C. E., Vol. xl, p. 471.

The experiments of 1897 upon the steel pipe utilized a length of about 4 400 ft.* In the wood pipe experiments a length of 2 710 ft. was used. The long section of wood pipe above Tunnel No. 7† was not used because the overflow at that point prevented securing static conditions in the pipe above.‡ When the present series of observations was made, it was found possible to stop the overflow by adjusting flash boards properly at the relief shaft and at the dam. Measurements of the loss of head in a length of about 22 700 ft. of the wood pipe above Tunnel No. 7 were therefore made. In addition, a limited number of experiments was made upon the portion of wood pipe used in 1897, as well as a new series upon the steel pipe. A velocity of flow, materially higher than in the preceding series, was secured.

Pressure Measurements.

Six pressure stations, located at the ends of the three portions of pipe above mentioned, were occupied. For convenience of reference, these stations have been designated by the numbers 1, 2, 3, 4, 5 and 6, beginning at the lower end of the steel pipe. Mercury gauges were used at all stations except No. 6, at the upper end of the long section of wood pipe. At this point the pressure was so small that a water piezometer was used.

At Stations Nos. 4 and 5 were placed the gauges used in 1897, and their description need not be repeated here. The only change made in these gauges was the attachment of fixed scales for the purpose of reading the position of the mercury in the reservoirs. The gauges used at Stations Nos. 1, 2 and 3 were open manometers similar to the others, but of a modified design.

In the former experiments, it was found difficult to make the gauges absolutely mercury-tight under the high pressure existing at the power-house, and this difficulty was carefully guarded against in planning the new series. The new gauges were made stronger than the old, and were tested at the highest pressure under which they were to be used. Means were also provided for determining fluctuations in the level

* The length was 4 427 ft. during a part of the series, and 4 367 ft. during the remainder.

† For plan and profile, showing the position of this tunnel, see paper by Henry Goldmark, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. xxxviii, p. 246.

‡ Static readings of the gauges could, of course, be dispensed with if the difference of level between the gauges were known with sufficient exactness. To determine this difference with the requisite accuracy by running a line of levels would have involved such an amount of labor and time as to put this method out of the question.

of the mercury surface in the manometer reservoir, so that, even if leakage of mercury occurred, the observation of the height of the mercury column would not be vitiated. For this purpose the gauge was provided with a glass tube placed vertically at the side of the reservoir and communicating with it at top and bottom (Fig. 1), and by the side of this tube was placed a fixed scale. The position of the

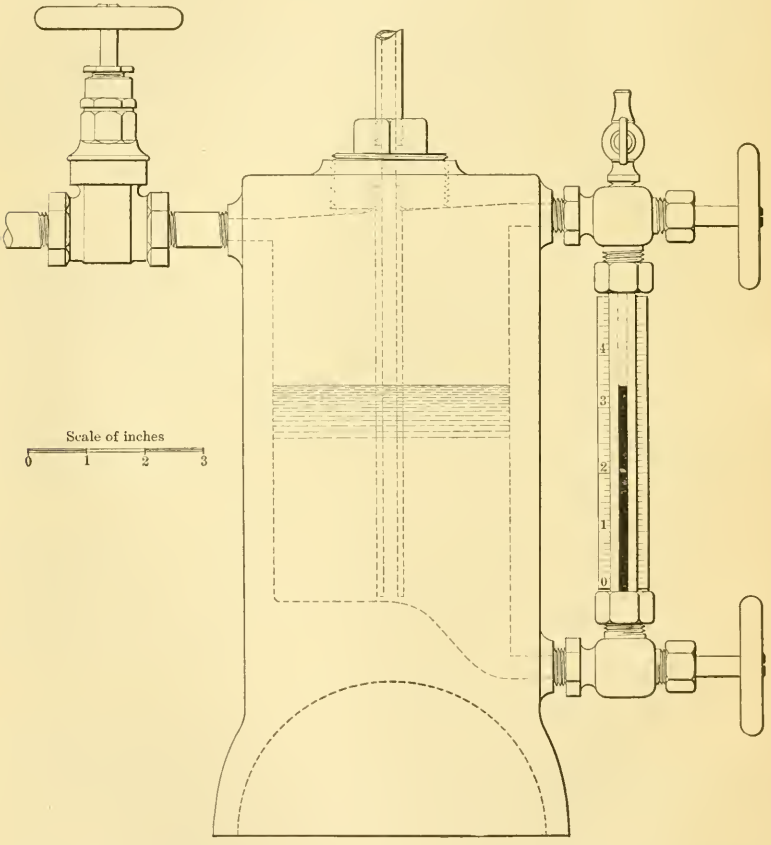


FIG. 1.

top of the mercury in this tube could be read with the same degree of precision as that of the top of the mercury column in the open tube The scales used were in all cases graduated to hundredths of a foot, and the third decimal place was estimated in taking readings.

As in the previous experiments, the long mercury column at Station No. 1 was provided with a water jacket carrying running water, to insure

a uniform temperature. At other stations the temperature was determined by thermometers placed beside the mercury columns.

The specific gravity of the mercury used, in terms of water from the pipe, was determined for the authors in the chemical laboratory of the Ogden Sugar Company, through the courtesy of the Superintendent, Mr. H. T. Dyer. The value adopted was 13.57.*

The pipe leading from the pressure section was in every case given a continuous upward inclination to the point of communication with the manometer reservoir, and was provided with a blow-off cock at the highest point, for the purpose of keeping this connecting pipe free from air. In the case of Manometer No. 1, this pipe was necessarily of considerable length, but the water in it could be completely changed by opening the blow-off for a few seconds. The vitiation of the results by air would require a sufficient accumulation to completely fill the cross-section of the pipe. It is reasonably certain that no such accumulation occurred in any piezometer. There was, in fact, no indication at any time that any important amount of air was carried by the water.

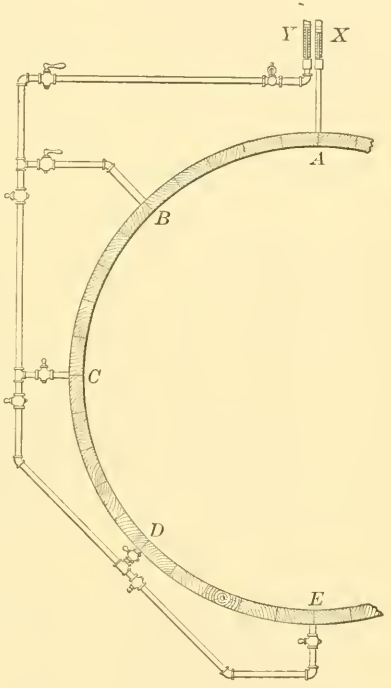


FIG. 2.

The attachment of piezometers to the main pipe was made in the same way as in the former series of observations,† except in the case of the water piezometer at Station No. 6. At this section the pipe was tapped at five points, *A*, *B*, *C*, *D*, *E* (Fig. 2). Two vertical glass tubes

* This value is believed to be reliable to within one-twentieth of 1 per cent. In reducing the former series of experiments, a value of 13.6 was used, the authors believing themselves to be justified in assuming, without an experimental determination, that this value was correct to the degree of accuracy demanded by the nature of the experiments. The difference between 13.57 and 13.6 would mean a difference of less than one-fourth of 1% in the values found for the loss of head. The authors have yet to learn of any experiments on flow in large pipes which can claim such a degree of accuracy as this figure represents, and do not claim such accuracy for their own experiments.

† *Transactions*, Am. Soc. C. E., Vol. xl, p. 475.

X and Y were used, the former communicating with the pipe at the single point A, the latter arranged to communicate with any one or more of the tubes running to B, C, D, E. The object of this arrangement was to test whether the indication of the piezometer was affected by the position of the point of attachment. For this purpose, simultaneous readings were taken of the water columns X and Y, the latter being in communication with the pipe at any one, or any combination, of the points B, C, D, E. The results showed a small difference between the reading of X and Y, the former being in all cases a little the higher. The oscillations of the two columns (which were somewhat rapid and not simultaneous) prevented a precise determination of the amount of this difference and of its variation with the velocity of flow. The best observations showed a difference of 0.02 ft. with a velocity of 4.7 ft. per second. No change in the reading Y appeared to be produced by changing the combination of points of communication with the pipe. It would seem, therefore, that the difference in the readings of X and Y was due to some accidental circumstance affecting the connection at A. The reading Y was used in all cases in the observations for determining loss of head in the pipe.

Measurement of Rate of Discharge.

The rate of discharge was determined as before, by attaching difference-gauges to the two Venturi meters. The two difference-gauges were placed side by side, so that both could be read by a single observer (Fig. 6, Plate V.). The connecting pipes were laid on a continuous up-grade from the pressure sections to the gauges, blow-off valves being placed at the summits to insure freedom from air.

Loss of Head in Meters.

The difference-gauges were triple, showing pressure-differences for three sections: upper section of Venturi, throat of Venturi, and full-sized section below Venturi. The pressure-difference for the first and third sections thus showed the loss of head between those points.

Programme of Tests.

In carrying out the observations, it was necessary to secure simultaneous observations of the rate of discharge and the pressure at each of the points between which the loss of head was to be computed. For this purpose, observers stationed at the several gauges in use took

FIG. 1. MANOMETER NO. 1. (1899.)

FIG. 2. MANOMETER NO. 2. (1899.)

FIG. 3. MANOMETER NO. 3. (1899.)

FIG. 4. MANOMETER NO. 5. (1899.)

FIG. 5. MANOMETER NO. 6. (1899.)

FIG. 6. DIFFERENCE GAUGE. (1899.)

readings at short intervals (generally one minute or less), throughout a period previously agreed on. During each period it was aimed to maintain a uniform rate of discharge. Because of fluctuations in the consumption of power by customers, the amount of water used by the wheels could not always be kept as constant as was desired, and it was necessary in most cases to use the average of readings, which varied somewhat during the interval in question. An interval of 20 minutes was adopted as the standard period covered by an "observation," and in most cases a nearly uniform flow was maintained during four consecutive 20-minute periods. The readings obtained during each 20-minute period were averaged, giving one "observation;" and, when the variation in the flow was small during several consecutive observations, these were combined to form a "group." The amount of difference between the observations of any group may be seen by reference to Table No. 1 (Plate VI), to be explained below. At the beginning of each 20-minute interval the blow-off valves were opened for a sufficient time to insure freedom from air in the gauges and in the connecting pipes.

Reduction of Observations.

The method of reducing the observations was somewhat different from that used before;* the difference, is however, in form rather than in substance.

(*1.*) *Reduction of Manometer Reading to Equivalent Water Column.*— Each manometer observation was reduced to an equivalent water-piezometer reading. For this purpose a zero or datum level is assumed for each gauge, the position of which is arbitrary, but which it is convenient to take as the zero of the lower fixed scale.

The data furnished by a manometer observation are the following: Height of top of mercury column above zero of upper scale; height of mercury surface in reservoir above zero of lower scale; temperature of mercury column. Each of these quantities is found by averaging the readings taken throughout an observation period. When the pressure was very unsteady, the readings were plotted before averaging; a comparison of the plotted results obtained from the different gauges being of assistance in the selection of the exact readings to be used. In most cases the readings were sufficiently steady to make plotting unnecessary.

* *Transactions*, Am. Soc. C. E., vol. xl., p. 480.

The vertical distance between zeros of the two scales is a known constant for each gauge. Adding to this the upper reading and subtracting the lower reading, the result is the actual vertical length of the mercury column. This must be reduced to an equivalent column at a standard temperature. Strictly, this temperature should be that of the water in the portion of pipe under experiment; practically, it makes little difference what temperature is selected as the standard, except that variations in the water temperature must be taken into account, if of sufficient amount to affect the results materially. When the experiments were begun, the temperature of the water was in the neighborhood of 10° Cent., and this was chosen as the standard. The reduction to this standard temperature is made by applying the factor $1 - 0.00018\,(T - 10)$, T being the temperature of the mercury column in degrees Cent., and 0.00018 being a sufficiently exact value of the coefficient of expansion of mercury within the range of temperature found in the experiments.

The mercury column is reduced to water by multiplying by the specific gravity of mercury, which, as previously stated, was found to be 13.57. To the height of water, thus computed, must be added the lower scale reading, the zero of this scale being taken as the fixed piezometer datum.

Changes of temperature of the water in the pipe must be considered next. The difference in level of the two pressure stations on the steel pipe is about 300 ft. The difference between the pressures at these two stations will be changed appreciably by a change of even a few degrees in the temperature of the water. Thus, the coefficient of expansion of water in the neighborhood of 10° Cent. is very nearly 0.000084. A change of 4° in the temperature (say from 8° to 12°) would therefore cause the relative level of piezometers at the two points to change by about $300 \times 4 \times 0.000084 = 0.1$ ft. (very nearly). Observations of the temperature in the waste flume below the power-house showed, during the period covered by the experiments, a variation between 9.5° and 14° Cent. It is apparent, therefore, that this cause probably had an appreciable influence upon the gauge readings at Stations Nos. 1 and 2.

From observations of the temperature in the flume, an estimate was made of the mean temperature of the water in the steel pipe during each observation, and the corresponding correction was applied

PLATE VI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XLIV, No. 879.
MARX, WING AND HOSKINS ON FLOW OF WATER.

TABLE No. 1.

to the reduced water column. Thus, if the water temperature is $t°$ Cent., the piezometer column, at 10°, computed as above, is multiplied by the factor $1 + 0.000084\ (t - 10)$.

This correction was applied only in case of Manometers Nos. 1 and 2. In the case of the wood pipe it is of comparatively little importance, because of the much smaller slope of the pipe. Moreover, although the flume temperature may give a fairly reliable indication of the variations of temperature in the steel pipe, it would be hopeless to attempt to secure any reliable estimate of the changes of temperature in the long section of wood pipe.

(2.) *Computation of Loss of Head.*—The loss of head between any two stations, due to a given velocity of flow, may be found by determining the difference between the piezometer columns at the two stations, and comparing it with the like difference under static conditions.* Thus, if z denotes the difference between simultaneous values of the piezometer heights at the two stations, and Z is the value of z under static conditions, $Z - z$ is the loss of head between the two stations. The datum of reference must remain constant for each piezometer, but its actual position is arbitrary. If the two sections of pipe have unequal diameters, the velocity-head must be added to the piezometer height in every case; but, with a uniform pipe, the velocity terms disappear from the value of the loss of head.

(3.) *Computation of Rate of Discharge.*—From a difference-gauge observation, the rate of discharge is computed in the usual way. The velocity through the throat of the Venturi is given by the formula $v = k' \sqrt{2\,g\,H}$, in which H is "head on Venturi," and the coefficient k' is the product of the friction coefficient k (in the notation of the previous paper), and the coefficient depending upon the areas of the upper section and throat of the Venturi. The values of k' for the Ogden meters, as furnished by the manufacturers were given in the former paper.†

General Results.

From each observation-group, the values of the following quantities have been computed: Loss of head per thousand feet; coefficient c in the Chezy formula $v = c \sqrt{rs}$; and "coefficient of roughness"

* In the previous paper (*Transactions*, Am. Soc. C. E., Vol. xl, p. 481), the method used was substantially the same; but the loss of head was first computed in mercury and then reduced to water, while in the present case each separate manometer observation was reduced to an equivalent water piezometer reading. This method was adopted for the reason that one of the gauges was a water piezometer, and its readings required no reduction.

† *Transactions*, Am. Soc. C. E., Vol. xl, p. 482.

n in Kutter's formula. These have been tabulated, and the values of c, n and loss of head per thousand feet are also shown graphically, being plotted with mean velocity of flow r as abscissas. Mean curves are drawn, representing each of these quantities as a function of the velocity, and, from the mean curves, generalized tables are computed. In these tables are included values of the friction coefficient f in the formula

$$H' = f \frac{l}{d} \frac{v^2}{2g},$$

H' being total loss of head in length l of pipe. These values are computed from those of c, from the relation

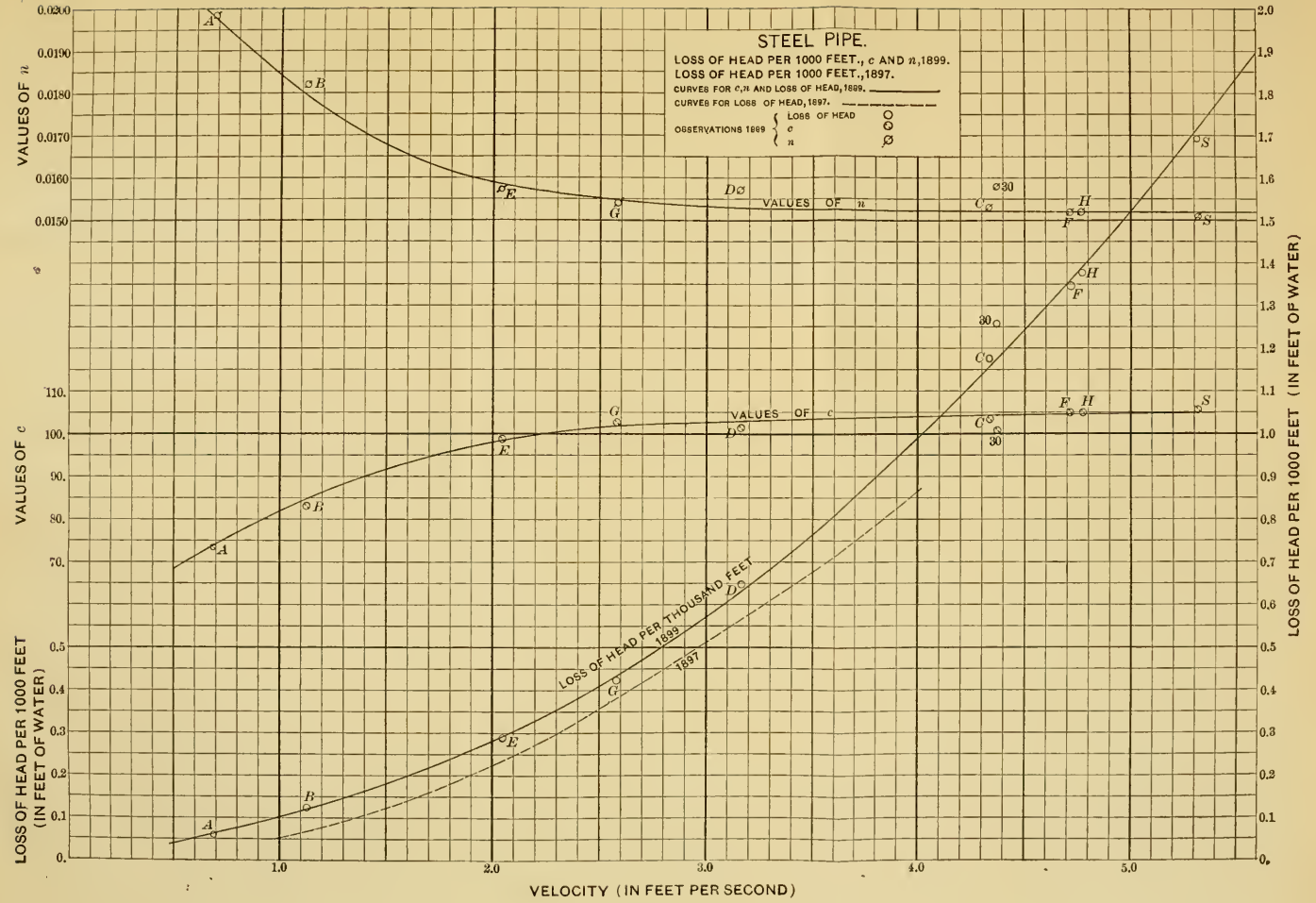
$$f = \frac{8g}{c^2}.$$

II.—Results of the Experiments.

Record of Observations.

The experimental data for the entire series of observations are shown in Table No. 1 (Plate VI). The observations are numbered in chronological order, the numbers being given in Column 2 of the table. The total number is 84; but No. 3 was rejected because it showed so great a discrepancy when compared with Nos. 2 and 4, in which the rate of discharge had the same value as in No. 3. In nearly every case the period of an observation was 20 minutes, though the actual interval during which readings were taken was less than this by from 2 to 5 minutes, because of the time occupied in opening the blow-off valves at the beginning of each period. The date and time of each observation are given in Column 3.

Columns 4, 5, 6, 7 and 8 contain the data obtained from the five mercury gauges at Stations Nos. 1, 2, 3, 4 and 5. In each case, the following quantities are entered: Upper scale reading (U), lower scale reading (L), temperature of mercury column (T). In the case of Manometers Nos. 1 and 2, the temperature of the water in the steel pipe (t) is also given. The constant distance between zeros of upper and lower scales is in each case entered at the top of the column. The sub-column headed "Mer." gives the actual height of the mercury column, and the sub-column "Mer. Reduc." gives the height of the mercury column corrected for temperature. In the case of Manometers Nos. 1 and 2 there is a double temperature correction, account being taken of both mercury and water temperatures.


STEEL PIPE.
LOSS OF HEAD PER 1000 FEET., c AND n,1899.
LOSS OF HEAD PER 1000 FEET.,1897.
CURVES FOR c,n AND LOSS OF HEAD,1899.
CURVES FOR LOSS OF HEAD,1897.
OBSERVATIONS 1899
LOSS OF HEAD
c
n
VALUES OF n
VALUES OF c
LOSS OF HEAD PER THOUSAND FEET
1899
1897
VALUES OF n
VALUES OF c
LOSS OF HEAD PER 1000 FEET (IN FEET OF WATER)
LOSS OF HEAD PER 1000 FEET (IN FEET OF WATER)
VELOCITY (IN FEET PER SECOND)
0.0200
0.0190
0.0180
0.0170
0.0160
0.0150
110.
100.
90.
80.
70.
2.0
1.9
1.8
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.
1.0
2.0
3.0
4.0
5.0
A
B
E
G
D
C
30
F
H
S

The method of applying the temperature correction may be explained by reference to a single observation, as No. 18. The data entered for Manometer No. 1 are as follows: Upper reading, 2.214; lower reading, 0.033; constant distance between zeros of scales, 31.300; mercury temperature, 10.8° Cent.; water temperature, 11.5° Cent. The actual height of the mercury column is, therefore, $31.300 + 2.214 - 0.033 = 33.481$ ft. To reduce to an equivalent column at 10° Cent. there must be subtracted $33.481 \times 0.00018 \times 0.8 = 0.005$ ft., 0.00018 being the coefficient of expansion of mercury. The correction for water temperature, already explained, has for convenience been applied to the mercury column before reduction to equivalent water column. In Observation No. 18 the temperature of the water in the steel pipe is 1.5° higher than the standard temperature of 10° Cent. The corresponding correction to the mercury column is $33.481 \times 0.000084 \times 1.5 = 0.004$ ft., 0.000084 being taken as the coefficient of expansion of water at temperatures near 10° Cent. The correction for mercury temperature is negative, while that for water temperature is positive; hence the reduced height of mercury column ("Mer. Reduc.") is $33.481 + 0.004 - 0.005 = 33.480$ ft.

It should be remarked that the constant distance between zeros need not be determined with great accuracy. Its only importance is in the computation of the temperature correction.

In Column 9 is given, for each gauge, the height of the "equivalent water piezometer." The datum of reference for each gauge is, as already remarked, arbitrary. If the zero of the lower scale is taken as datum, the equivalent water piezometer height is determined by multiplying the reduced mercury column by 13.57 (the relative specific gravity of the mercury at 10° Cent.), and adding the lower scale reading. Thus, referring to Observation No. 48, the data for Manometer No. 3 are

7.136 = reduced height of mercury column;
0.006 = lower scale reading.

Hence the height of the equivalent water piezometer is $7.136 \times 13.57 + 0.006 = 96.84$ ft.

In the case of Manometer No. 1, the reduced mercury column is in every case diminished by 30 ft. before the reduction to "equivalent water piezometer." This simplifies the logarithmic computation, and is allowable because it amounts merely to shifting the datum of refer-

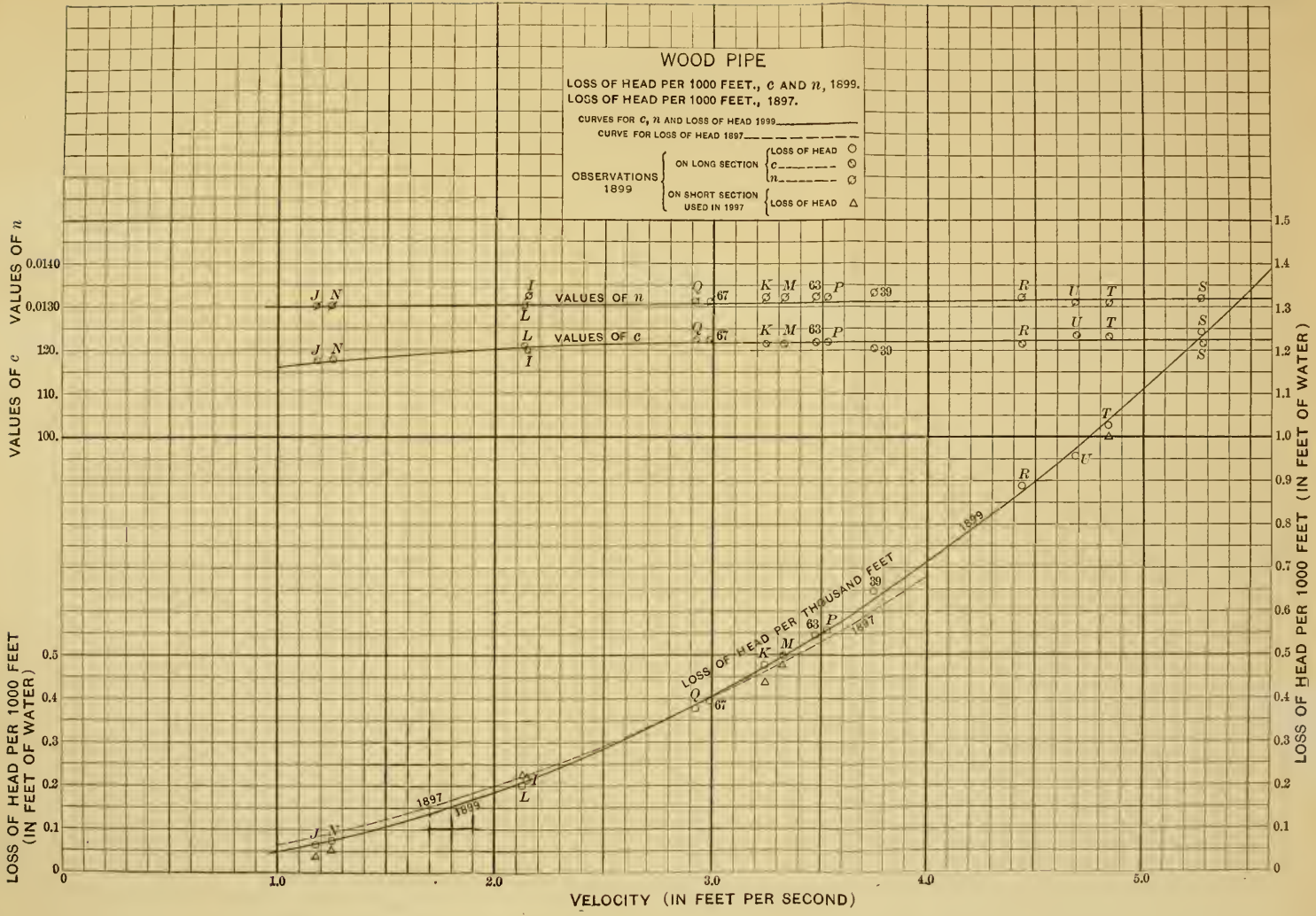
ence by 407.10 ft. (the water equivalent of 30 ft. of mercury). The values of "Mer. Reduc." for Gauge No. 2 have in like manner been diminished by 8 ft.

The sixth sub-column under Column 9 gives the reading of Gauge No. 6. The "equivalent water piezometer" is in this case the actual reading of the gauge. Since the height of water column is only 5 or 6 ft., the temperature correction for this gauge is inappreciable. About 20° change of temperature of the column would be required to change the reading by 0.01 ft.

Sub-column 7, under Column 9, gives the stage of water in the reservoir above the dam. It is obtained by measuring down from a fixed point on the masonry, and the values are therefore given the minus sign. Many of the values entered are obtained by interpolation between observations made an hour or more apart. No special device was used to secure great accuracy in these readings, and in some cases they are affected by an uncertainty of perhaps 0.05 ft. because of waves. These readings are not used in estimating pipe coefficients, but were taken for the purpose of estimating the loss of head in the entire pipe line.

For computing the loss of head between two manometer stations, the difference between simultaneous piezometer columns at these stations is taken. Values of this difference for each of the three lengths of pipe experimented on, and also for Gauges Nos. 4 and 5, are given in Column 10. These values are found in each case by taking differences between the numbers in the corresponding sub-columns of Column 9. The process of computing loss of head is completed by subtracting these differences from the corresponding static difference for each pair of manometers compared. The results are given in Column 11.

Static readings could be secured only during one hour each week, when the water-wheels were stopped. Three sets of static readings were taken, the results being recorded as Observations Nos. 1, 34 and 76. When Observation No. 1 was made, only Manometers Nos. 1 and 2 were ready for use. Observation No. 34 includes readings of all manometers. In Observation No. 76 readings were taken on Manometers Nos. 1, 2, 5 and 6. The static difference for the steel pipe (1–2) was thus observed three times; but as Observation No. 34 on the steel pipe was not regarded as very satisfactory at the time, the static dif-


WOOD PIPE
LOSS OF HEAD PER 1000 FEET., c AND n, 1899.
LOSS OF HEAD PER 1000 FEET., 1897.
CURVES FOR c, n AND LOSS OF HEAD 1899
CURVE FOR LOSS OF HEAD 1897
OBSERVATIONS 1899
ON LONG SECTION
LOSS OF HEAD
c
n
ON SHORT SECTION USED IN 1897
LOSS OF HEAD
VALUES OF n
VALUES OF c
LOSS OF HEAD PER THOUSAND FEET
1899
1897
VALUES OF n
VALUES OF c
LOSS OF HEAD PER 1000 FEET (IN FEET OF WATER)
LOSS OF HEAD PER 1000 FEET (IN FEET OF WATER)
VELOCITY (IN FEET PER SECOND)
0.0140
0.0130
120.
110.
100.
0
0.1
0.2
0.3
0.4
0.5
1.0
1.1
1.2
1.3
1.4
1.5
1.0
2.0
3.0
4.0
5.0

ference adopted for Gauges Nos. 1 and 2 was based upon Observation Nos. 1 and 76, which differ by only 0.01 ft. The two values are 13.19 and 13.18; when the computation is carried to the third decimal place the mean of these is 13.186, and value adopted is 13.19. It will be seen that Observation No. 34 gives a value differing from this by 0.06 ft. If this had been used in computing the mean, the result would have been changed by only 0.02 ft. Only one determination of the static difference 3–4 was made. For 5–6 there were two determinations, giving values 49.49 and 49.55, the mean of which (49.52) was adopted.

In Column 11 are entered the values of the total loss of head and of the loss per thousand feet, for each of the three lengths of pipe 1–2, 3–4, 5–6. At the top of each sub-column is given the length of pipe between manometers. A sub-column is also given for values of the loss between Gauges Nos. 4 and 5, from which the loss due to Tunnel No. 7 may be estimated.

In Columns 12, 13, 14 and 15 are entered the results of the discharge observations. The sub-column headed "gauge" for each meter gives the difference between the heights of the throat and up-stream mercury columns of the Venturi meter difference-gauges. "Head on Venturi" is equal to this difference multiplied by e—1, e being the specific gravity of the mercury, 13.57. From this gauge reading and the known dimensions of the meter the rate of discharge is computed, as already explained. The remaining columns, giving rate of discharge for each meter, total rate of discharge, and mean velocity of flow in the steel and in the wood pipe, need no explanation.

In most cases several consecutive observations (usually four) were made with as nearly constant conditions of discharge as possible. When the actual variation was small, such observations were combined into a "group." These groups are designated by letters, which are given in Columns 1 and 17 of Table No. 1 (Plate VI). Table No. 2 shows values of mean velocity and loss of head, as obtained by averaging the results of each group of observations. The other quantities given in this table will be referred to later.

Steel Pipe Results.

The steel pipe results for each observation-group are given in Table No. 2, Columns 3, 4, 5 and 6.

TABLE No. 2.

1	2	3	4	5	6	7	8	9	10	11	12	13
	Steel Pipe. Man. 1-2.					Wood Pipe. Man. 3-4.		Wood Pipe. Man. 5-6.				Total Loss. Man. 4-5.
Group.	Numbers of observations.	Velocity in feet per second	Loss of head per 1 000 ft. in feet.	c.	n.	Velocity in feet per second.	Loss of head per 1 000 ft. in feet.	Velocity in feet per second.	Loss of head per 1 000 ft. in feet.	c.	n.	Total loss in feet.
	1	0.	−0.009	.....	.....	.....	.....	.....	.....	.....	.....	.....
A..	2, 4	0.689	0.059	73.4	0.0198	.....	.....	.....	.....	.....	.....	.....
B..	5, 6, 7, 8, 9	1.120	0.121	83.1	0.0182	.....	.....	.....	.....	.....	.....	.....
C..	10, 11, 12, 13	4.339	1.177	103.1	0.0153	.....	.....	.....	.....	.....	.....	.....
D..	14, 15, 16, 17	3.157	0.645	101.3	0.0157	.....	.....	.....	.....	.....	.....	.....
E...	18, 19, 20, 21	2.043	0.286	98.5	0.0157	.....	.....	.....	.....	.....	.....	.....
F..	22, 23, 24, 25	4.713	1.343	104.8	0.0152	.....	.....	.....	.....	.....	.....	.....
G...	26, 27, 28, 29	2.577	0.423	102.1	0.0154	.....	.....	.....	.....	.....	.....	.....
	30	4.375	1.258	100.6	0.0158	.....	.....	.....	.....	.....	.....	.....
H..	31, 32, 33	4.779	1.379	104.9	0.0152	.....	.....	.....	.....	.....	.....	.....
	34	0.000	−0.000	.....	.....	0.000	0.000	0.000	0.001	.....	.....	.....
I...	35, 36, 37, 38	.....	.....	.....	.....	2.144	0.219	2.144	0.213	119.5	0.0132	0.137
	39	.....	.....	.....	.....	3.750	.....	3.750	0.644	120.2	0.0133	.....
J...	40, 41, 42, 43	.....	.....	.....	.....	1.175	0.038	1.175	0.066	117.3	0.0130	0.147
K..	44, 45, 46, 47	.....	.....	.....	.....	3.239	0.434	3.239	0.474	121.1	0.0132	0.225
L..	48, 49, 50, 51	.....	.....	.....	.....	2.126	0.224	2.126	0.206	120.7	0.0130	0.110
M..	52, 53, 54, 55	.....	.....	.....	.....	3.324	0.474	3.324	0.498	121.2	0 0132	0.212
N..	56, 57, 58, 59	.....	.....	.....	.....	1.244	0.054	1.244	0.074	117.7	0.0130	0.095
P..	60, 61, 62	.....	.....	.....	.....	3.527	.....	3.527	0.557	121.7	0.0132	.....
	63	.....	.....	.....	.....	3.474	.....	3.474	0.542	121.4	0.0132	.....
Q..	64, 65, 66	.....	.....	.....	.....	2.920	.....	2.920	0.378	122.2	0.0131	.....
	67	.....	.....	.....	.....	2.987	.....	2.987	0.396	122.2	0.0131	.....
R..	68, 69, 70, 71	4.473	.....	.....	.....	4.439	.....	4.439	0.889	121.2	0.0132	.....
S...	72, 73, 74, 75	5.320	1.696	105.3	0.0151	5.279	.....	5.279	1.242	121.9	0.0132	.....
	76	0.000	+0.009	.....	.....	0.000	.....	0.000	-0.001	.....	.....	.....
T..	77, 78, 79, 80	.....	.....	.....	.....	4.845	1.003	4.845	1.027	123.0	0.0131	0.402
U.	81, 82, 83, 84	.....	.....	.....	.....	4.694	.....	4.694	0.959	123.3	0.0131	.....

The same results are shown graphically in Plate VII. In this diagram mean curves are drawn to represent the relation between mean velocity of flow and each of the three quantities, loss of head per thousand feet, c and n. The curve of loss of head given by the experiments of 1897 is also shown for the purpose of comparison. The later series covers a range of velocities materially greater than the earlier, the greatest value of the mean velocity being 5.32 ft. per second in the observations of 1899 as against 3.85 ft. per second in the series of 1897.

A reference to Fig. 6 of the previous paper * shows that the portion of the curve corresponding to the higher velocities was based largely upon five observations made under conditions of falling press-

* *Transactions*, Am. Soc. C. E., Vol. xl., p. 495.

TABLE No. 3.—Steel Pipe. Generalized Results.

v.	H'.		c.		f.		n.	
	1897.	1899.	1897.	1899.	1897.	1899.	1897.	1899.
1.0	0.055	0.100	110.0	81.6	0.0212	0.0387	0.0132	0.0184
1.5	0.121	0.177	111.0	92.0	0.0207	0.0303	0.0140	0.0167
2.0	0.220	0.277	110.0	98.0	0.0212	0.0267	0.0144	0.0159
2.5	0.356	0.405	108.0	101.3	0.0221	0.0251	0.0147	0.0155
3.0	0.510	0.570	108.0	102.4	0.0221	0.0245	0.0147	0.0154
3.5	0.673	0.765	110.0	103.2	0.0212	0.0242	0.0145	0.0153
4.0	0.863	0.987	111.0	103.8	0.0207	0.0238	0.0143	0.0153
4.5	.....	1.237	.....	104.3	.....	0.0236	.....	0.0152
5.0	.....	1.516	.....	104.7	.....	0.0234	.....	0.0152
5.5	.....	1.824	.....	105.0	.....	0.0233	.....	0.0152

ure. It seems possible, in the light of the later results, that this portion of the curve is somewhat too low. But making all reasonable allowance for the uncertainty in the values found, it appears that there has been some decrease in the carrying capacity of the steel pipe.

By interpolation, using the mean curves of Plate VII, values of the several quantities may be found for velocities 1, 1.5, etc., feet per second. These generalized values of H', c, f and n are shown in Table No. 3, together with the corresponding values from the observations of 1897.

The Chezy coefficient c shows an increase with the velocity. For velocities above 2.5 ft. per second, this increase, however, is slow, and the upper limit to the value of c would appear to be not greater than 106.

The Kutter coefficient n shows, for low velocities, a decrease with increasing velocity. For higher velocities n approaches a limiting value of about 0.0152.

It must be remembered that the law of variation of c and of n for low velocities cannot be regarded as reliable, because the values found for these quantities are affected very materially by small errors in the measured loss of head.

Wood Pipe Results.

The wood pipe results for all the observation-groups are shown in Table No. 2. For the shorter length of pipe (3–4) the table gives only the velocity and the loss of head per thousand feet (Columns 7 and 8). For the longer portion, values of c and n are also given.

TABLE No. 4.—Wood Pipe. Generalized Results.

v.	H'.		c.		f.		n.	
	1897.	1899.	1897.	1899.	1897.	1899.	1897.	1899.
1.0	0.066	0.049	100.0	116.0	0.0257	0.0191	0.0150	0.0130
1.5	0.123	0.106	110.0	118.7	0.0212	0.0183	0.0141	0.0130
2.0	0.200	0.184	115.0	119.9	0.0194	0.0179	0.0137	0.0131
2.5	0.292	0.284	119.0	120.8	0.0181	0.0176	0.0133	0.0132
3.0	0.400	0.404	122.0	121.4	0.0173	0.0175	0.0131	0.0132
3.5	0.527	0.548	124.0	121.7	0.0167	0.0174	0.0130	0.0132
4.0	0.678	0.712	125.0	122.0	0.0165	0.0172	0.0128	0.0132
4.5	.....	0.898	.....	122.2	.....	0.0172	.....	0.0132
5.0	.....	1.105	.....	122.4	.....	0.0172	.....	0.0132
5.5	.....	1.335	.....	122.5	.....	0.0171	.....	0.0132

The coefficients c and n and the loss of head per thousand feet are shown graphically in Plate VIII, being plotted as functions of the mean velocity of flow. This diagram shows the observations on both portions of the wood pipe, the two sets of points being marked differently.

As the number of observers was too small to permit the simultaneous occupation of all the six manometer stations, it was thought best to devote special attention to the long section of wood pipe, and to the steel pipe. The number of observations upon the shorter length of wood pipe was therefore limited, their main value being to serve as a check upon the results of 1897, and to show whether any material difference exists between the coefficients for the two portions of wood pipe. The mean curves shown in Plate VIII are intended to represent the results for the long section only. For the purpose of comparing with the previous experiments on the lower section, the mean curve of loss of head as found from the experiments of 1897 is also shown.

The generalized results for the upper section of wood pipe are given in Table No. 4, the values of H', c, f and n being obtained from the mean curves of Plate VIII. The table includes also values of the same quantities for the lower section of pipe, as found from the experiments of 1897.

Loss of Head Due to Tunnel No. 7.

Tunnel No. 7 is 108.7 ft. long, and is unlined, its cross-section being approximately 9 ft. square. Manometer Stations Nos. 4 and 5

were on opposite sides of this tunnel, the former being 23 ft. from its west end and the latter 44 ft. from its east end. Simultaneous readings of the gauges at these two stations furnish date for estimating the loss of head due to the tunnel.

TABLE No. 5.—Tunnel 108.7 Ft. Long. 9 Ft. Square.

Velocity in wood pipe, in feet per second.	Loss between manometers 4 and 5, in feet.	Loss in 67 ft. of wood pipe, in feet.	Loss due to tunnel, in feet.	Equivalent length of wood pipe, in feet.
1.0	0.044	0.003	0.041	835
2.0	0.110	0.012	0.098	532
3.0	0.190	0.027	0.163	405
4.0	0.290	0.048	0.242	340
5.0	0.430	0.074	0.356	322

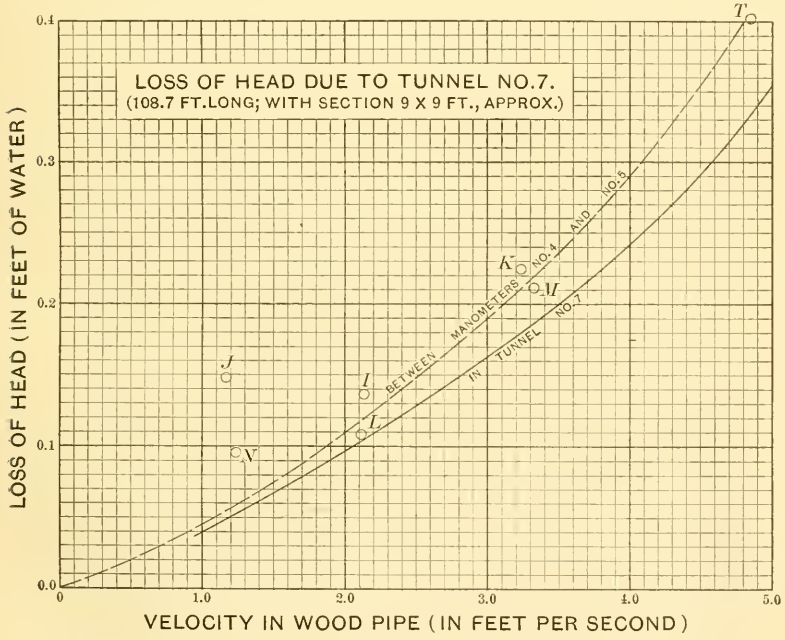


Fig. 3.

The total loss between Gauges Nos. 4 and 5 is given in Table No. 1 (Plate VI), Column 11, and the condensed results are shown in Table No. 2, Column 13. These results are shown graphically in Fig. 3, in which is also drawn a curve to represent the relation between the total

TABLE No. 6.

1	2	3	4	5	6	7	8	9	10
No.	South Meter.		North Meter.		Group.	South Meter.		North Meter.	
	$\frac{H}{e-1}$	$\frac{H''}{e-1}$	$\frac{H}{e-1}$	$\frac{H''}{e-1}$		$\frac{H}{e-1}$	$\frac{H''}{e-1}$	$\frac{H}{e-1}$	$\frac{H''}{e-1}$
2.....	0.038	0.006	.....	.....	*A*.....	.....	.....	.....	.....
3.....	0.038	0.007	.....	.....	*A*.....	0.038	0.007	.....	.....
4.....	0.038	0.007	.....	.....	*A*.....	.....	.....	.....	.....
5.....	0.101	0.020	.....	.....	*B*.....	.....	.....	.....	.....
6.....	0.101	0.019	.....	.....	*B*.....	.....	.....	.....	.....
7.....	0.101	0.019	.....	.....	*B*.....	0.101	0.019	.....	.....
8.....	0.101	0.019	.....	.....	*B*.....	.....	.....	.....	.....
9.....	0.101	0.019	.....	.....	*B*.....	.....	.....	.....	.....
10.....	0.659	0.112	0.186	0.030	*C*.....	.....	.....	.....	.....
11.....	0.668	0.113	0.184	0.029	*C*.....	0.659	0.111	0.186	0.030
12.....	0.660	0.112	0.187	0.031	*C*.....	.....	.....	.....	.....
13.....	0.648	0.108	0.186	0.030	*C*.....	.....	.....	.....	.....
14.....	0.582	0.099	0.027	0.003	*D*.....	.....	.....	.....	.....
15.....	0.556	0.094	0.027	0.002	*D*.....	0.546	0.091	0.027	0.003
16.....	0.537	0.090	0.027	0.003	*D*.....	.....	.....	.....	.....
17.....	0.510	0.082	0.027	0.003	*D*.....	.....	.....	.....	.....
18.....	0.345	0.058	.....	.....	*E*.....	.....	.....	.....	.....
19.....	0.331	0.056	.....	.....	*E*.....	0.341	0.058	.....	.....
20.....	0.331	0.056	.....	.....	*E*.....	.....	.....	.....	.....
21.....	0.358	0.061	.....	.....	*E*.....	.....	.....	.....	.....
22.....	0.816	0.138	.....	.....	*F*.....	.....	.....	.....	.....
23.....	0.815	0.137	.....	.....	*F*.....	0.813	0.137	.....	.....
24.....	0.812	0.136	.....	.....	*F*.....	.....	.....	.....	.....
26.....	0.544	0.091	.....	.....	*G*.....	.....	.....	.....	.....
27.....	0.544	0.091	.....	.....	*G*.....	0.545	0.092	.....	.....
28.....	0.546	0.095	.....	.....	*G*.....	.....	.....	.....	.....
31.....	0.506	0.087	.....	.....	*H*.....	0.506	0.087	.....	.....

loss of head between gauges and the mean velocity of flow in the wood pipe. Upon this curve are based the generalized results given in Table No. 5.

The total length of wood pipe between the two gauges is 67 ft.; the loss of head due to this length, assuming the loss per thousand feet to have values as given in Table No. 4, is computed and entered in Table No. 5. Deducting this from the total loss between gauges, the loss due to the tunnel is found. The total tunnel loss is made up of loss at entrance, loss at outlet, and loss due to resistances in the tunnel itself. It is not possible to estimate these losses separately so as to determine the value of c for the tunnel. The curve showing total loss due to tunnel for different values of velocity in the wood pipe is given in Fig. 3.

The last column of Table No. 5 shows the length of wood pipe which would produce the same loss as the tunnel.

Loss of Head in Venturi Meters.

A limited number of observations was made on the loss of head due to the Venturi meters. The first and third tubes of each of the triple difference-gauges communicated with the pipe at points where the values of the mean velocity were equal, the diameter being 54 ins. at each section. The observed pressure-difference between these sections, therefore, was due wholly to the loss of head in the intervening portion of the stream.

It should be stated that this observed loss is not all chargeable to the meter. The portion of the pipe between the two pressure sections included, besides the converging and diverging portions of the meter, about 7 ft. of 54-in. riveted pipe. The diverging pipe below the throat is constructed of riveted plates, and a gate-valve, occupying

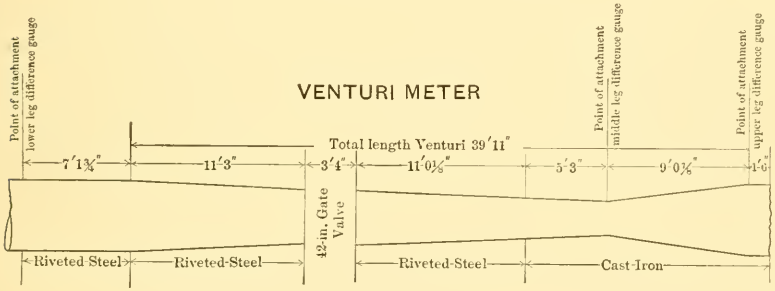


FIG. 4.

about 40 ins., is situated in this portion of the pipe. The whole arrangement is shown in Fig. 4. Doubtless, the greater part of the observed loss of head occurred between the throat and the down-stream pressure section. While the loss in the diverging pipe may be properly charged to the meter, since this is a necessary part of the apparatus, this is not true of the loss in the 7 ft. of 54-in. pipe below, nor of the loss due to the gate. The points of attachment of the difference-gauge tubes were the same in these experiments as in the previous series.

The results of the observations on the loss of head in the meters are shown in Table No. 6. "Head on Venturi" is denoted by H, and "loss of head" by H''. The specific gravity of the mercury being e, the difference between the up-stream and throat mercury columns is

$\frac{H}{e-1}$, and the difference between the up-stream and down-stream columns is $\frac{H''}{e-1}$. These are the quantities observed directly, and the observed values are given in the table. These values are averaged in groups, and the average values are entered in Columns 7, 8, 9 and 10. The observation numbers and the letters designating groups agree with those in Table No. 1 (Plate VI).

The results are plotted in Fig. 5. As in the previous series of observations,* the plotted points fall very nearly on a straight line, indicating a relation expressed by the equation $H'' = a\,H$, with a constant value of a. The value found for a is 0.169, while in 1897 the value was 0.149. The lines corresponding to both these values are shown in Fig. 5.

It thus appears that the loss of head between the two sections compared increased 13.4% between August, 1897, and June, 1899. This is on the assumption that the loss of head in the meter proper (*i. e.*, between the upper section and the throat) has remained constant; for unless this is true a given value of "head on Venturi" corresponds to different values of the rate of discharge in the two series of experiments.

How nearly correct this assumption is, there are no means of knowing. It is probable, however, that the change in the loss in the converging part of the meter is slight.

It may be pointed out that an increase of 13.4% in the loss of head, for any given velocity, agrees well with the increase observed in the 72-in. steel pipe. This may be verified by a comparison of the 1897 and 1899 curves of loss of head in Plate VII.

The continued courtesy of C. K. Bannister, M. Am. Soc. C. E., Chief Engineer of the Union Power Company, and of the Board of Directors, enabled the writers to duplicate and extend the series of observations made in 1897. To them and to Mr. L. S. Boggs, Electrical Engineer, the writers are under great obligations for giving *carte blanche*, so far as they were able to do so without interfering with the running of the plant. As the writers were short-handed at times, the work could not have been carried through in the time at their disposal if the Superintendent of the Power Company, Mr. C. E. Crocker,

* See Figs. 13 and 14 of previous paper.

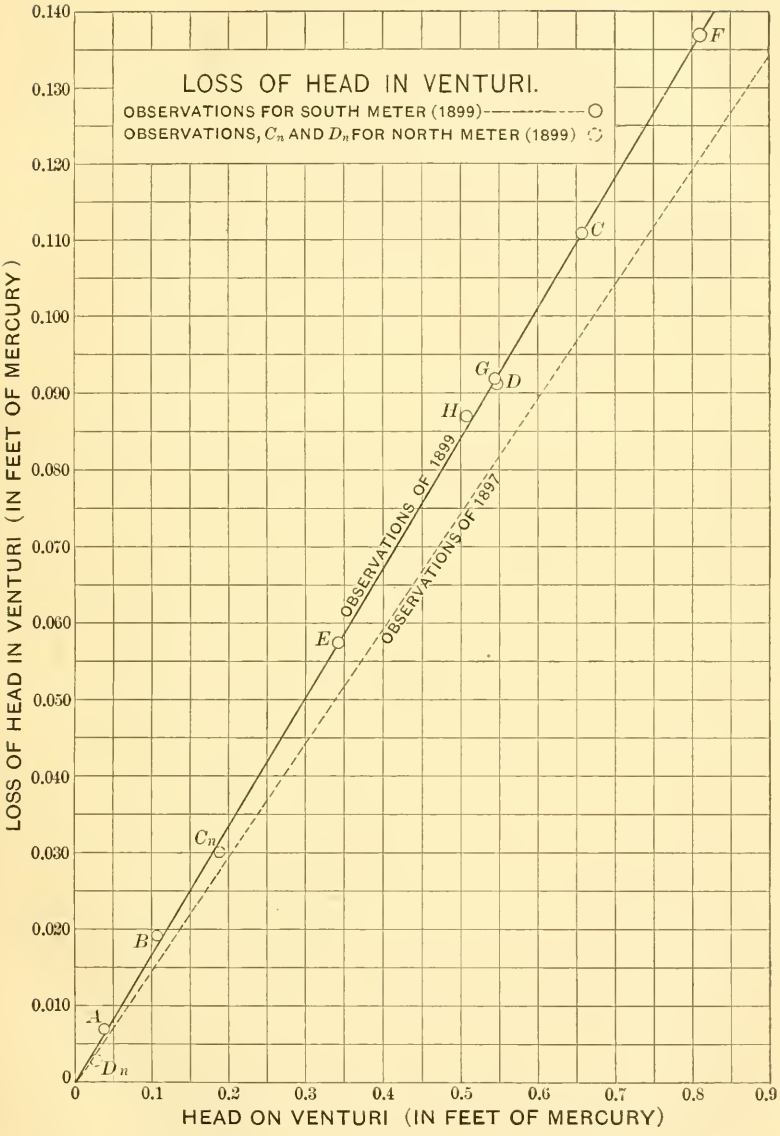

LOSS OF HEAD IN VENTURI.
OBSERVATIONS FOR SOUTH METER (1899)
OBSERVATIONS, C_n AND D_n FOR NORTH METER (1899)
OBSERVATIONS OF 1899
OBSERVATIONS OF 1897
LOSS OF HEAD IN VENTURI (IN FEET OF MERCURY)
HEAD ON VENTURI (IN FEET OF MERCURY)
0.140
0.130
0.120
0.110
0.100
0.090
0.080
0.070
0.060
0.050
0.040
0.030
0.020
0.010
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
A
B
C
D
E
F
G
H
C_n
D_n

FIG. 5.

had not kindly assisted in taking readings. He also regulated the discharge so as to give, so far as practicable, the desired velocities. For his assistance and co-operation the writers desire to express their sincere thanks. Their former assistant, Mr. L. B. Spencer, Assistant Engineer, Oregon Short Line, came from Salt Lake on two occasions to help, and to him also the authors owe thanks. During the entire period of their stay they were, furthermore, assisted by their colleague, Professor J. C. L. Fish, who has added to their obligations by preparing a number of the diagrams which accompany this paper.

DISCUSSION.

Mr. Kuichling.

E. KUICHLING, M. Am. Soc. C. E.—One of the most interesting features of this paper is the comparison of the observations made by the authors in 1897 on the large riveted-steel pipe with those made by them in 1899. From this comparison, as exhibited in Table No. 3, they conclude that during the period of two years there has been some decrease in the carrying capacity of this pipe. A similar, although much smaller, reduction is also noticeable in the case of the large wooden pipe, for velocities of 3 ft. and upward, as shown in Table No. 4. The data submitted are very important to hydraulicians, as it rarely happens that the same line of pipe is tested by the same competent and careful observers at different periods of time.

In the speaker's opinion, such a decrease in carrying capacity is always to be expected, and, so far as he has been able to ascertain from the records of reliable gaugings made by others, as well as by himself, it has always occurred, especially in the case of cast and wrought-iron or steel pipes. Two principal causes therefor can be assigned, viz., the gradual formation of nodules of rust, or "tubercles," and the accretion of various organic growths on the inner surface of the conduits. Either of these developments must obviously increase the resistance to the flow of the water in the pipe, and hence also diminish the discharge. The first-named cause is necessarily excluded in the case of a wooden conduit, but in an iron or steel pipe both causes may exist simultaneously.

Rust, or tuberculation, in a water pipe is the result of the action of the water upon the iron, wherever exposed, by defects or abrasions of the protective coating. The area of the imperfection may be very small or even microscopic in size, but its site will sooner or later be manifested by the appearance of a yellowish brown pimple, which gradually increases in size and ultimately assumes relatively large dimensions by spreading laterally over the adjacent sound coating material. Two or more neighboring accumulations of this kind frequently merge into a single large crust, and, when it is forcibly removed, the few minute sources of the supply of hydrated iron oxide are often readily discernible. This oxide evidently attracts or unites with some of the earthy and mineral matter in the water, and the resulting mixture becomes quite coherent, besides adhering more or less strongly to the inner surface of the pipe. In this manner a tubercle is slowly formed, and when the earthy matter preponderates over the iron oxide, it may also become the seat of an extensive organic growth, thereby increasing its size and the degree of obstruction to the flow.

As already stated, the organisms which may lodge and develop in a

Mr. Kuichling. pipe are of various kinds, and may be of either animal or vegetable character, but adapted to the absence of sunlight and free air. They generally grow on the sides and top of the conduit, and rarely on the bottom, as they appear to dislike the subsidence of fine silt which is apt to occur when the velocity of the water is either greatly reduced, or entirely stopped by entrainment in the branches. Several species of polyzoa, spongilla and algæ, or fungi, were found easily by the speaker in searching for them in dark chambers of reservoir gate-houses and also in the conduits and distributing pipes of several different water-works, but in the latter they were usually restricted to the upper portion of the line or to the vicinity of the source or reservoir. In some cases, however, these growths were found in pipes at a distance of several miles from the head.

An instructive instance of this kind was observed recently in the new conduit of the Rochester, N. Y., water-works. The northern section thereof is about 9 miles long, and consists of about 1 000 ft. of 36-in. cast-iron pipe, beginning at Rush Reservoir, followed by 38-in. riveted-steel pipe for the remainder of the distance to the city. A number of gaugings of the flow of this section, and measurements of the loss of head in the cast-iron and steel pipes separately, have been made as carefully as was possible, by the same corps of assistants, since its completion in 1894, and the essential results of these experiments are given in Table No. 7.

TABLE No. 7.—Results of Gaugings of the Northern Section of the New Conduit of the Rochester, N. Y., Water-Works, by the Same Parties, at Different Times between October 17th, 1895, and November 10th, 1899.

No. of gauging.	Date.	Duration of experiment, in hours.	Observed hydraulic grade in the 36-in. cast-iron pipe.	Observed hydraulic grade in the 38-in. riveted-steel pipe.	Observed mean velocity in the 36-in. cast-iron pipe, in feet per second.	Observed mean velocity in the 38-in. riveted-steel pipe, in feet per second.	Deduced coefficient (c) in $v = c\sqrt{rs}$ for the 36-in. cast-iron pipe.	Deduced coefficient (c) in $v = c\sqrt{rs}$ for the 38-in. steel pipe.	Mean age of the conduit in service, in years.
	1895.								
1..	Oct. 17...	5.35	0.0013876	0.0015866	4.204	3.876	129.45	109.35	1.28
2..	Oct. 26...	5.22	0.0015066	0.0016137	4.239	3.908	125.25	109.34	1.30
3..	Nov. 7...	6.38	0.0015034	0.0016180	4.234	3.904	125.25	109.07	1.36
	1897.								
4..	July 28..	10.77	0.0022774	0.0016250	4.128	3.806	99.22	106.11	3.06
5..	Nov. 11..	7.92	0.0048187	0.0015315	4.045	3.730	66.84	107.11	3.35
6..	Nov. 19..	7.08	0.0048535	0.0015334	4.023	3.709	66.24	106.46	3.37
	1898.								
7..	June 4...	8.95	0.0043408	0.0015655	4.034	3.719	70.23	105.65	3.91
8..	Dec. 21...	8.00	0.0037591	0.0015531	4.026	3.712	75.32	105.86	4.47
	1899.								
9..	July 25...	8.00	0.0034455	0.0015349	4.084	3.765	79.79	108.00	5.04
10..	Nov. 10..	8.00	0.0032585	0.0015424	4.077	3.759	81.93	107.58	5.34

Mr. Kuichling.

It should be noted that the actual mean diameter of the 36-in. cast-iron pipe is 3.0406 ft., as deduced from four measurements of the bore of each separate piece before being set in place, while that of the inside courses of the 38-in. riveted-steel pipe is 3.1667 ft.; also that the corresponding cross-sectional areas are, respectively, 7.2612 and 7.8758 sq. ft., and the lengths between the end piezometers, respectively, 1 094.3 and 45 393.9 ft. The difference in level between the piezometer vessels was determined instrumentally, while the volume discharged by the pipe in the given periods of time was computed from the observed fall in the water surface of the large reservoir at the head of the section, the area for different elevations being known. The water to supply the piezometers was taken in each case from a standard corporation cock screwed squarely into the cast-iron pipe at or near its top. Furthermore, at the distributing reservoir in the city, the steel conduit terminates with about 250 ft. of 36-in. cast-iron pipe, but the length thereof which is embraced, up to the lower piezometer, is included in the 1 094.3 ft. mentioned.

When the extremely low value of the coefficient c for the 36-in. pipe in Gauging No. 5 became known, it was assumed that an error had been made, and the work was repeated a few days later, with practically the same result, as shown in Gauging No. 6. Special care having then been taken to avoid errors of observation, it was conjectured that the reduction was due to a profuse growth of aquatic organisms in this part of the line, such as was found in the 24-in. effluent pipe of the old conduit at the same locality when a part of it was removed in 1894 for connection with the new pipe. A corroboration of the accuracy of the observations in Gaugings Nos. 5 and 6 was also afforded a few months later by Gauging No. 7.

To ascertain whether this conjecture was correct, an examination of the interior of the pipe was made in September, 1898. The flanged head of a 36-in. special casting in the new gate-house at Rush Reservoir was accordingly removed, and the 36-in. cast-iron pipe was entered and traversed by an assistant for a distance of about 450 ft., while the 38-in. steel pipe was entered at a manhole 3 498 ft. north of the gate-house and traversed for 100 ft. or more in both directions. Owing to the existence of several depressions in the line between these points of entry, and the necessity of soon restoring the new conduit to active service, no further investigations of the intermediate parts of the pipe were made.

On entering the 36-in. cast-iron pipe in the gate-house, extensive organic growths were seen attached to the top and sides, while the lower part was considerably tuberculated. These growths soon increased in magnitude until they formed an almost continuous lining for the entire distance traversed, with frequent large masses hanging down from the top. A number of such pieces were noticed, which were

Mr. Kuichling. more than 1 ft. square, stretching like curtains across the upper part. The lining, in general, resembled a dense fibrous mat from 1 to 2 ins. thick, and the glossy pitch coating of the pipe was visible only in a few places. Further examination showed that these growths consisted mainly of two species of polyzoa and one of fresh-water sponge, which were identified by Professor Charles W. Dodge, of the University of Rochester, N. Y., as *Plumatella*, *Paludicella* and *Myenia fluviatalis*. At the point where the 38-in. steel pipe was entered, the organic growths and tuberculation had greatly diminished in both size and extent, as only a few scattering patches, from 2 to 8 ins. in diameter, and not exceeding $\frac{1}{2}$ in. thick, were found, along with some small tubercles. The latter were on the lower part of the pipe in the immediate vicinity of the manhole, and doubtless resulted from the abrasion of the coating by the shoes of the workmen during construction. Elsewhere, the coating was clean, hard and glossy, and apparently as sound as when first applied. Several attempts were made to obtain photographs of the interior of the pipe by flash light at the localities mentioned, but unfortunately all the negatives, on being developed, proved to be dim and imperfect. The foregoing verbal description must therefore suffice to convey an approximate notion of its condition, and hydraulicians will doubtless require no further reasons for the low values of the coefficient c in the Chezy formula which were found from the gaugings relating to the comparatively short section of cast-iron pipe.

From this examination of the conduit, it will be seen that the luxuriant organic growths were limited to perhaps 1 000 ft. or more of the upper part of the line, while the formation of tubercles is possible for its entire length. The reason for such restriction of the extent of the former is obviously found in the fact that the food supply for the organisms, which is contained in the water, is correspondingly limited and becomes practically exhausted beyond a certain distance from the source; hence, in the lower part of the line no organic growths will probably be found. It may therefore be concluded that the gradual diminution of the carrying capacity of a pipe by the development of aquatic organisms on its interior is dependent on the quality of the water, and that when the proper food supply is scanty, little trouble from this cause will be found in a long conduit. On the other hand, there is no such natural limitation to the formation of rust in an iron pipe which is not provided with an absolutely perfect protective coating; and as such a coating has hitherto been practically unattainable, it follows that a reduction of discharge due to this second cause must reasonably be anticipated.

Returning to the case of the Ogden conduit, which was put in service early in 1897, and whose present interior condition is unknown, it is very probable that while no appreciable quantity of organic growths may exist in the steel pipe which forms the lower

portion of the long line, there may yet be sufficient tuberculation to account for the reduced discharging capacity which was found by the authors. Two years' time is sufficient for the purpose, when it is remembered that the asphaltic coating was applied by hand, and that numerous imperfections in such work are inevitable, notwithstanding the most rigid inspection. Proof of this conjecture, however, is lacking, and it is therefore to be hoped that the authors will at some future time supplement their present valuable contribution to hydraulics by a third series of gaugings, in which an opportunity to examine the condition of the interior of the conduit will be afforded. Mr. Kuichling.

Another interesting point in the paper is the considerably increased loss of head in the Venturi meters, upon whose registration dependence is placed for obtaining the discharge and mean velocity in the conduit. The authors have assumed that the loss of head in the meter proper has remained constant, but were unable to verify this assumption. Should it be found by experiment that a change in the frictional resistance of the two tapering sections of riveted pipe which constitute the meter will modify the empirical constant used in computing the discharge, as is by no means improbable, the given numerical values of the Chezy coefficient c will also require suitable modification; and hence, in conducting another series of gaugings, the means for testing the registration of the meters should also be included.

G. C. Whipple, Assoc. M. Am. Soc. C. E.—The growth of microscopic organisms in water is, certainly, an important subject, because such growths have a marked effect upon the flow in pipes. These organisms are likely to grow in pipes whenever the water flowing therein contains a sufficient amount of food to nourish them. Mr. Whipple.

The polyzoa, fresh-water sponge, etc., are sedentary forms which grow on the sides of the pipe, and must have their food carried to them. If the water contains the right kind of food to nourish them it is quite likely that they will thrive in any kind of a pipe. The whole subject is chiefly a question of food supply.

For example, in Newton, Mass., which is supplied with a ground-water containing practically no microscopic organisms, an examination of the pipes showed that there were very few polyzoa. In Boston, on the other hand, where the water contains these microscopic forms, many of the pipes were found to be filled with them.

The presence or absence of these organisms can be accounted for, to a great extent, by the microscopical character of the water itself. It is possible that the water flowing in the Ogden pipe-line may be so free of the smaller microscopic organisms that none of the sedentary organisms are present in any part of the pipe-line, while the water may be of such a character as to cause tuberculation of the steel pipe, without having any such chemical action on the wooden pipe.

Mr. Whipple. It would be interesting to know whether the authors made any examination of the quality of the water flowing through the pipe, and whether any opportunity was offered them to examine the pipe-line with reference to the presence of polyzoa, fresh-water sponge and other similar organisms.

Mr. Meem. JAMES COWAN MEEM, Assoc. M. Am. Soc. C. E. (by letter).—In comparing the steel pipe tests for 1897 and 1899, it is noted that with clean pipes, in 1897, the value of the coefficient of roughness n was not only much lower, but was also much more uniform than in 1899, when the pipe had undoubtedly become tuberculated with rust and growths. It is undoubtedly a valuable increment to our knowledge to find that with tuberculated pipes the value of n seems to decrease in proportion to the rise of velocity, below, and up to 3 ft. per second; while, for practical purposes, it may be considered as constant under the same conditions for clean pipes under all velocities.

This latter point is borne out by the 1899 experiments on the wood-stave pipes, which show a constant value for n for all velocities; and it is further borne out by some crude experiments made by the Sewer Department of Brooklyn, in 1896, on the 15-ft. brick sewer on Fourth Avenue, which also showed a practically constant value for n under varying velocities.

The comparison between the 1897 and 1899 experiments on the wood-stave pipe shows a marked decrease in the value of n after two years' use. This doubtless shows that organisms require more, as Mr. Whipple suggests, than that their food be brought to them, but that they must also have a "lodging place," which is probably furnished in the steel and iron pipes by the commencement of rust, and points to the further fact that wood pipe seems to wear smoother with usage. Arthur L. Adams,* M. Am. Soc. C. E., states that the growths in wood pipe are not found to exist where the pipe is constantly full and under pressure. It is probable, then, that the presence of decay only in this pipe furnishes the nucleus about which these organisms grow.

Comparing these experiments with those noted or made by Mr. Adams,† it is seen that the value of n is much lower in the stave pipes of smaller sizes. Thus, in the 72-in. pipe in the paper it is 0.0131, while in the experiments by Mr. Adams on 14-in. and 18-in. pipes, $n = 0.010$ for the 18-in., and 0.0107 to 0.011 in the 14-in. pipe. This confirms substantially what the writer has long believed to be true, that in pipes of small diameters, all else being equal, the value of n is lower than for pipes of large diameter. Comparison of the experiments referred to, on the 15-ft. brick sewer in Brooklyn (which though crudely made showed the value of n to be between 0.013 and 0.015), with some other experiments‡ made by T. C. Hatton, M. Am. Soc. C. E., of Wil-

* *Transactions*, Am. Soc. C. E., Vol. xli, p. 84.

† *Transactions*, Am. Soc. C. E., Vol. xli, p. 55.

‡ Published, the writer believes, in *The Engineering Record*.

mington, Del., on small pipes (in which the value of n was shown to be materially lower than 0.013) partially confirms this conclusion, although not decisively or definitely enough to be of value, as they were not compared under the same conditions. Mr. Meem.

The summary of these deductions is as follows:

(*a*) In smooth pipes of the same size and kind the value of n is practically constant under all velocities.

(*b*) In tuberculated pipes under all high velocities the value of n is constant, for the same sizes and conditions; while

(*c*) In tuberculated pipes, for the same conditions, under low velocities, the value of n increases inversely with the velocity.

(*d*) In clean pipes the value of n probably decreases with the diameter of the pipe, other conditions being the same.

In assigning the value of n, in estimating or calculating the velocities in proposed pipes and conduits, it is, of course, beyond the range of possibility to test the proposed pipes under the exact conditions of usage; or to find experiments fitting each case exactly. The writer, therefore, proposes a factor of safety, as in construction, which may be used at the discretion of the engineer. As our tables and diagrams are usually calculated for a value of 0.015 for n, we may take this as a basis on which the following factors of safety are suggested for adoption:

(1) For steel-riveted or cast-iron pipes, calculated to run under low velocities, or which are inaccessible for cleaning, add 20% to the total discharge in making the final calculations for sizes.

(2) For steel-riveted or cast-iron pipes, under high velocities, or which are accessible for cleaning, add 10 per cent.

(3) For brick, concrete and pipe conduits, under all velocities, and of over 3 ft. in diameter, or wood-stave pipe alternately wet and dry, add nothing.

(4) For small pipes of all kinds except iron or steel, and wood-stave pipes of over 3 ft. diameter, and running full at all times, deduct 10 per cent.

(5) For wood-stave pipes of less than 3 ft. diameter, constantly full and under pressure, deduct 20 per cent.

Gardner S. Williams, M. Am. Soc. C. E. (by letter).—The authors are to be congratulated upon having performed the most accurate series of experiments yet published upon wooden-stave pipe. Their observations upon the riveted pipe do not, however, appear to be quite so reliable, either from unsuspected sources of error in the methods and means of observation, or, much more probably, on account of conditions in the pipe line itself which prevent the general application of the results obtained from it; so that, while the conclusions which may be drawn from them may not be suitable for common use, the experiments themselves, for the particular case in hand, may Mr. Williams.

Mr. Williams. be in every way as accurate and deserving of confidence as those upon the upper section of stave pipe, which the writer believes will compare favorably with the best pipe experiments yet published, and which appear to be of wide application. The reasons for this differentiation of the results will be discussed later. Before proceeding with this question, however, there are other points in the paper which should be considered.

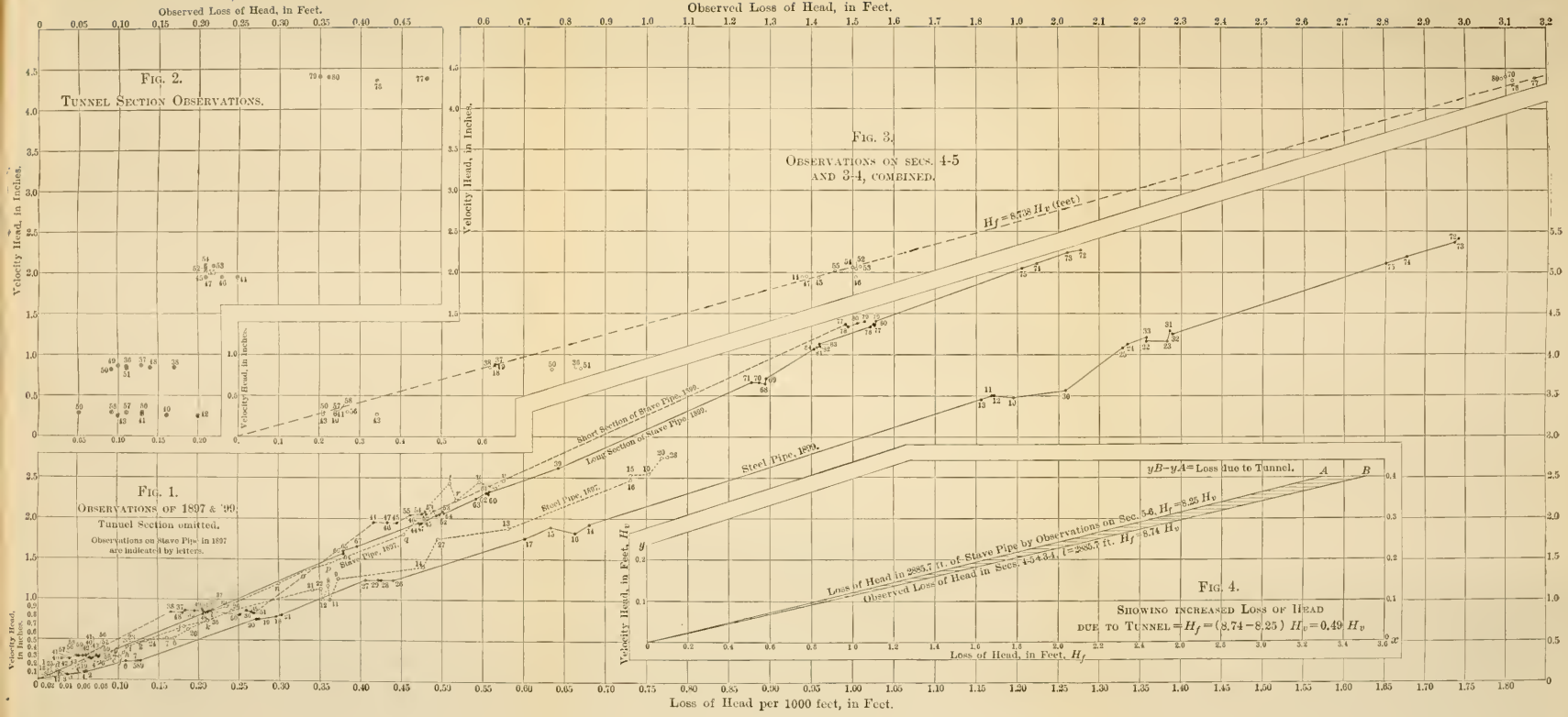
The authors (page 37) argue that in order to have the gauge readings vitiated by accumulations of air in the connections, there must be present a quantity sufficient to fill the cross-section of the pipe completely. It appears to the writer that any mixture of air and water must have a less specific gravity than water alone, and hence, if even minute bubbles of air be present in a connection rising to a gauge, the effect will be to cause the gauge to read high. As this very principle has long been utilized in the old forms of air-lift pumps, it does not seem necessary to discuss it further than to say that if the connecting pipe slopes downward to the gauge, the result will be a low reading, and that while very few, if any, of the present observations give evidence of such a source of error, the fact remains that observations may be affected by air which does not fill the cross-section of the pipe.

On the same page the authors refer to observations with piezometers connected at various points in the circumference, from which they appear to conclude the correctness of the very interesting theoretical discussion of this subject presented in their former paper.*

On this point the writer has to submit that he knows of experiments with very delicate apparatus for measuring the pressures at different points of the circumference of a pipe in which water was flowing, which showed that the pressures not only are different at different points, but that they change sign relatively to each other as the velocity changes, one becoming alternately greater and less than the other at the opposite extremity of the same diameter. As it is expected that these experiments will shortly be presented to the Society, with some others on the flow of water in pipes, the writer does not wish to go into detail here, but it seems to him entirely safe to conclude that the hypotheses assumed by the authors in their former discussion were contrary to fact, and that in the present case, had they had more delicate apparatus, they would probably have found a difference in the readings at the several points of the circumference.

In reference to the general results, it is always desirable to have some criterion by which to judge of the accuracy of experiments as they are published, aside from that of the personality and reputation of the observers; and the writer has for some time been in the habit of applying to his own observations the following:

* *Transactions*, Am. Soc. C. E., Vol. xl, p. 522.


Observed Loss of Head, in Feet.
Velocity Head, in Inches.
Fig. 2.
Tunnel Section Observations.
Observed Loss of Head, in Feet.
Velocity Head, in Inches.
Fig. 3.
Observations on secs. 4-5 and 3-4, combined.
$H_f = 8.75\ H_v$ (feet)
Short Section of Stave Pipe, 1899.
Long Section of Stave Pipe, 1899.
Steel Pipe, 1899.
Steel Pipe, 1897.
Fig. 1.
Observations of 1897 & '99.
Tunnel Section omitted.
Observations on Stave Pipe in 1897 are indicated by letters.
Velocity Head, in Inches.
Loss of Head per 1000 feet, in Feet.
$yB - yA$ = Loss due to Tunnel.
A
B
Loss of Head in 2885.7 ft. of Stave Pipe by Observations on Sec. 5-6, $H_f = 8.25\ H_v$
Observed Loss of Head in Secs. 4-5 4-3 4-1 = 2885.7 ft. $H_f = 8.74\ H_v$
Velocity Head, in Feet, H_v
Loss of Head, in Feet, H_f
Fig. 4.
Showing increased Loss of Head due to Tunnel = $H_f = (8.74 - 8.25)\ H_v = 0.49\ H_v$

Mr. Williams.

Assuming that the loss of head varies as the square of the velocity, the former must also vary as the first power of the velocity head; whence, in any series of experiments covering more than one velocity, if the velocity heads and the corresponding losses of head be used as co-ordinates, theoretically, the points so located should fall in a straight line which may be expected to pass through the origin; and the deviation of the points so plotted from such a line will indicate the relative merit of the several observations. It will at once be appreciated that while some form of curve can usually be made to pass through and fit fairly well a great many observations, it is not such an easy matter to fit a straight line to them.

This criterion has been applied by George H. Fenkell, Jun. Am. Soc. C. E., to nearly all the so-called standard pipe experiments hitherto published, and it is somewhat surprising to see how many supposedly reliable observations are nothing more than crude approximations. It is hoped that Mr. Fenkell's work may soon be given to the public in order that the records may be purged of a considerable number of experiments which should never have been published, and that the relative merits of others may be estimated properly.

An application of this criterion to the observations of the authors in 1897 showed that the high degree of accuracy claimed by them, based upon reductions by the method of least squares, was frequently imaginary.

In Fig. 1, Plate IX, the writer has plotted the observations presented in Table No. 1 (Plate VI) of the present paper, and also the data of the previous series of experiments. To avoid unnecessarily complicating an already rather intricate diagram, the straight lines to which the observations should conform have not been drawn, but if a fine thread be stretched along the line of observations for the long section of stave pipe, the excellence of the results will be at once apparent, and if the equation of the mean line be computed by summations of the ordinates and abscissas we obtain $H_f = 2.8643\ H_v$, where H_f is the loss of head and H_v is the velocity head, measured in feet.

Similarly, if the thread be stretched along the points for the short section of stave pipe, which are united by the fine broken line it will not only be seen that the inclination of the line is not the same as in the other case, but also that the points do not fit as well, and that, in fact, this series of observations appears to be very little better than those of 1897, which are indicated by letters and joined by the dotted lines.

Observations Nos. 40, 41, 56, 57, 58, 59, 37, 38, 48 and 44, seem to show too low a loss of head as compared with the velocity head, while 35, 36, 50 and 51, show the opposite condition. Excepting the four last named, it appears that in this section of pipe the observations at low velocity indicate a much lower resistance than corresponding observations on the longer section, a result which, while it does not

Mr. Williams. accord with the former observations on this section, as indicated by *A*, *B*, *C*, *D*, *E*, *G*, *H* and *K*, is seemingly correct.

Remembering this condition of affairs in Section 3–4, we now consider the Tunnel Section 4–5, the observations upon which are plotted in Fig. 2, Plate IX. By stretching the thread here it will be seen that the observations fail to have the semblance of conformity to the straight line, and it appears that all the observations at low velocities show abnormally high losses of head, exactly the opposite condition to that observed in the section next down stream, already considered. These facts seem to indicate that the piezometer at No. 4 gave a low reading, particularly at low velocities. Without attempting to account for this at present we will combine the total losses of head in Section 4–5 with the simultaneous ones in Section 3–4, and plot the combined result to the velocity heads in the stave pipe, which plotting is shown in Fig. 3, Plate IX. In this figure, the mean line obtained by summation of ordinates has been drawn, Nos. 42, 50, 36 and 51 being rejected. It is to be remarked that Nos. 36, 50 and 51, seem to contain errors of observation or reduction, but as they were grouped by the authors with others this point does not attract especial attention in their plottings, although quite evident here. The close approximation of the remaining observations to the straight line, except perhaps No. 46, is quite remarkable, although there is still a tendency for the low velocities to show excessive loss of head, which has no counterpart in the observation on the long section of stave pipe.

The following explanation of the phenomenon might not have been so obvious, but for the fact that a similar one was encountered recently by Messrs. E. C. Murphy and C. C. Torrance, their observations being corroborated by the writer, in the Hydraulic Laboratory of the College of Civil Engineering of Cornell University, where such precautions were taken as to leave no possibility of questioning the results.

All are familiar with the fact that when water flows from the bottom of a vessel to a pipe of smaller cross-section, a spiral motion is generated, as may be seen at any time in the ordinary washbowl. It is, therefore, aside from other proofs, quite safe to assume, when there is an abrupt change of section in a pipe line from a larger to a considerably smaller one, that a similar spiral motion will be set up, and it needs no extended argument to prove that the higher the velocity, the longer the pitch of the spiral, which latter will keep lengthening as the water flows on in a straight pipe, the particles tending continually to move more and more nearly in straight lines. It is well known, further, that velocity head is convertible into pressure head and *vice versa*. It becomes apparent, therefore, that if the pressure head be measured a short distance below a point where a reduction of cross-section of the flowing stream has taken place, we may expect to find that the true velocity of a particle of water, the flow at that point being in a spiral,

will exceed the rectilinear velocity in the direction of the axis, and will also exceed the true velocity further along the pipe where the pitch of the spiral has been increased, whence the pressure head at the up-stream point will be reduced by the excess of velocity head there, and it is entirely possible to conceive, what actually occurs in the Venturi meter, that the following length of pipe might show a gain of pressure rather than the loss to be expected from frictional resistances. This is, apparently, exactly what takes place in the Ogden pipe on which Piezometer No. 4 is 23 ft. down stream from the tunnel mouth where a contraction takes place from a section 9 ft. square to a 6-ft. circular one. The lower the velocity, the shorter the pitch of the spiral. Therefore, at low velocities, the ratio of the true or spiral velocity of the particles to the rectilinear velocity will be much greater than at high velocities; hence the pressure head will be much lower relatively to that at other points, as observed by the authors. The gradual straightening of the spiral results in readings farther down much more nearly in accord with those above and unaffected by the contraction. It may be remarked that this spiral motion, while apparently reducing the loss of head in the section below the contraction, must actually increase it; and a very interesting point is raised as to the rapidity with which the filaments straighten themselves out, or, in other words, how far below a contraction we must go to get a normal pressure reading. The fact that the lower observations, as plotted in Fig. 3, Plate IX, show an excessive loss of head, indicates that a distance of 2 733 ft. in the 6-ft. pipe is not entirely sufficient to hide its effects from observation with the mercurial measuring apparatus used here. How much farther the effect would be noticeable, with a water column or some more delicate apparatus, is an open question. Mr. Williams.

To determine the effect of the tunnel, it is evident that it is not proper to proceed as the authors have done and from the difference shown by Piezometers Nos. 4 and 5 take the computed loss in 67 ft. of stave pipe. The nearest to a correct proceeding, possible with the data at hand, is to take from the loss of head shown between Piezometers Nos. 5 and 3 the loss due to an equal length of stave pipe as deduced from the observations on the long section, when the effect of the tunnel will be seen to follow the same law as that of other resistances to flow, *i. e.*, to increase as the square of the velocity, and not in the manner indicated by the authors upon pages 48 and 49.

On Fig. 4, Plate IX, the result is shown graphically, the abscissas between the lines *O A* and *O B* showing the apparent increased loss of head due to the tunnel. From this it appears that the loss is about 6% greater with the tunnel as it is than it would be with the stave pipe continued through it.

It may be proper to call attention to the fact that in Figs. 1, 2 and 3, Plate IX, the velocity heads are plotted in inches of water, not

Mr. Williams. in feet, as the writer happened to have at hand a transformation curve for velocities to velocity heads in this unit. In Fig. 4, Plate IX, the velocity heads are plotted in feet.

Considering the steel pipe, it appears that the two groups of observations, Nos. 22, 23, 24, 25, 31, 32 and 33, and Nos. 72, 73, 74 and 75, fall very well upon the straight line through the origin, while the others show a higher loss of head than the line fitting the observations named. If we fit a line to the observations from Nos. 18 to 11, as plotted, the observations at the lowest velocities still show losses of head which are too high, so we have either to assume that the loss of head in riveted pipe does not vary as the square of the velocity, or else look for a cause for a low reading in Piezometer No. 1 or a high one in No. 2. The apparently close and accurate instrumental work done by the authors at the other gauges does not give much weight to the probability of errors of observation here, so that the cause is rather to be sought in the line itself. The data published seem hardly sufficient for the case, although the curvature in the line may possibly set up enough spiral flow to cause the lower gauge to read low. The authors themselves are best qualified to discuss this question. As to the possibility that the law in riveted pipe may be more than slightly different from that in cast-iron or wood, it may be said that the observations of Emil Kuichling, M. Am. Soc. C. E., on the Rochester conduits and of Clemens Herschel, M. Am. Soc. C. E., on the East Jersey Water Company's lines, do not indicate anything of the sort, nor do observations upon highly tuberculated pipes, nor, as pointed out previously, those on the tunnel section. The writer, therefore, has come to the conclusion regarding the riveted pipe work, already expressed, that although the observations may be equally accurate, the conditions are not such as to make the results as valuable as are those on the stave pipe.

From the apparent fact that the observations on the short section of stave pipe are influenced by the tunnel effects which are subject to quite wide variation, according as the flow is increased or decreased to produce the velocity under consideration, it seems rather questionable to attempt to determine the effect of age upon the capacity of the conduit by comparisons between the observations in 1897 and those of 1899 on this section. A similar criticism may be made as to the steel pipe, though to a somewhat more restricted degree.

The effect of spiral currents upon the registry of the meters is another interesting consideration. From the velocity head plottings, both in the stave pipe experiments and those on the riveted pipe, which were not simultaneous, it appears that there was a similar irregularity of results with velocity heads between 0.30 and 0.45 in. This, of course, points to a common cause, which seems most likely to be found in the meters. The position of the meters being such that

the water enters by a Y and a curve,* it is easy to conceive that, at some velocities the spiral motion generated at the Y may be reduced by the curve, and at others intensified there, and this might easily account for the irregularity referred to, as a greater or less spiral motion would reduce or increase the difference of head between the inlet and the throat, which seems to indicate that, after all, the Venturi meter, like many another type, is only absolutely reliable when used under exactly the conditions at which it was rated. Mr. Williams.

In this connection it may be well to add that when observations are plotted, as the writer has done in these cases, it is possible and proper to reject inaccurate ones, and to deduce coefficients from the points on the line rather than from the actual observations, thus eliminating, to a large extent, the individual inaccuracies. Having proceeded thus far, it only remains to drop the ancient Chezy and the modern Kutter formulas, and adopt one with an approximately logical foundation, to get the subject of hydraulics into a shape consonant with nineteenth century progress.

George W. Rafter, M. Am. Soc. C. E. (by letter).—This paper is an elegant sequel to the previous one, and presents, in compact form, data of considerable value to hydraulicians. The minute detail with which the authors have described their methods and appliances will be, without doubt, very satisfactory to those who find themselves unable to judge of the value of a paper when presented on the broad lines of useful results, without reference to the methods used in reaching them. The authors are to be congratulated on having given apparently everything necessary for intelligent judgment. Mr. Rafter.

The foregoing reflections are not in any way suggested by deficiencies in the paper, but refer to its minute fulness. As regards hydraulic methods and appliances, every possible question seems to have been answered, thus leaving one free to discuss the broad problem of increase of friction head with advancing age in large riveted steel, wood-stave or other water conduits, and without special reference to the detail of the present experiments, which, taken in conjunction with the authors' previous paper, may be assumed to illustrate the hydraulic conditions of the Ogden pipe in considerable detail.

Broadly, we may assume that many water conduits decrease somewhat in carrying capacity with advancing age. The paper indicates that for the Ogden pipe, this decrease is greater in steel pipe than in wood. Indeed, the wood pipe is shown to have increased in carrying capacity with advancing age. The reasons for these differences are not given, although it seems clear enough that an investigation of flow through a water conduit which reveals such marked changes in carrying capacity, as well as differences between two kinds of pipes, as are here indicated, ought to be accompanied by a study of the causes for such changes and differences.

* *Transactions*, Am. Soc. C. E., Vol. xxxviii, p. 279.

Mr. Rafter. Engineers sometimes err in not tracing out final causes, especially in cases like the present one where fields of knowledge, outside of engineering pure and simple, require to be traversed in order to reach an adequate solution. Nevertheless, the writer does not criticise the paper because of such omission, but merely mentions it as a suggestion for future work.

In 1891, when the writer presented his paper on "The Hydraulics of the Hemlock Lake Conduit of the Rochester, N. Y., Water-Works,"* drawing the conclusion, from certain discharge measurements made in 1890, that the high values of c in the expression $v = c\sqrt{rs}$, previously used in computing flow through large mains, were not justified by the facts, his conclusions as to the relatively low values of c really applying to riveted-steel conduits, were somewhat questioned. Time, however, is a great clarifier, and not only are the writer's views as to the lower values of c now universally admitted, but, since 1891, considerable energy has been expended in showing why, in large steel conduits of some age, it is impossible that any view other than the writer's could possibly apply. On this line, a theory of gradual deterioration has been built up, and many facts have been gathered, tending to substantiate it.

While the writer has no desire to controvert the view that many water conduits have decreased in carrrying capacity with age, he nevertheless wishes to point out that this result is by no means universal—the Ogden wood-stave pipe is a case in point—and that there are undoubtedly a considerable number of water conduits a long time in use, which are to-day discharging at full capacity substantially as much water as when first placed in service.

Speaking broadly, the evidence, as it now stands, apparently indicates that steel-plate conduits are particularly subject to decrease in carrying capacity with increased age. So far as the writer can determine, there seems to be a practical difficulty in coating the built-up-plate conduits, which, in some degree, militates against their usefulness. It is true that coated cast-iron pipes have also shown decrease in carrying capacity, but not, the writer concludes, to such an extent as the wrought-iron or steel-plate conduits.

As regards deficiencies in the coating, the writer is unable to add very much to his discussion of Mr. FitzGerald's paper on "Flow of Water in 48-In. Pipes," presented to the Society in 1896, and it is proposed, therefore, to give here a brief account of what may be termed the biological causes for the decrease in carrying capacity, referred to.

So far as present information goes, difficulties of this sort have been referred to two classes of animal life, namely, fresh-water sponges and fresh-water polyzoa. The fresh-water sponges have been introduced to the Society by Mr. FitzGerald in his two papers, "Spongilla

* *Transactions*, Am. Soc. C. E., Vol. xxvi, p. 13.

in Main Pipes "* and "Flow of Water in 48-In. Pipes," † but without giving any account of them. So far as the writer now recollects, aside from a brief reference in the discussion of his paper on "The Hydraulics of the Hemlock Lake Conduit of the Rochester, N. Y., Water-Works," the polyzoa have not been considered before this Society as the cause of decrease in carrying capacity of water conduits. Hence, the writer feels justified in giving a brief statement of the habits of the sponges and polyzoa. Mr. Rafter.

Knowledge of the American forms of fresh-water sponges was extended greatly by the monograph ‡ of Edward Potts, who has described and figured a considerable number of species, and his paper may be referred to as embodying practically all that we know about the American forms of these interesting animals.

Some of the fresh-water sponges may form incrustations like those described by Mr. FitzGerald in his paper entitled, "Flow of Water in 48-In. Pipes," although, thus far, only one or two species have been certainly identified as offending in these particulars, as, for instance, *Spongilla lacustris* at Boston, and *Meyenia fluviatalis* at Rochester. It is possible that on further study other forms of fresh-water sponge may be found to be giving rise to this trouble, although, aside from one or two forms, such supposition is rendered slightly improbable because close conduits do not seem to be the natural home of many species.

According to Mr. Potts, fresh-water sponges do not differ in constitution and general appearance from marine sponges, except that the fresh-water forms are characterized by the presence of certain seed-like bodies called gemmules or statoblasts, and which are in effect eggs from which new colonies issue. These gemmules are not found in marine sponges.

The gemmules are nearly spherical and about $\frac{1}{50}$ in. in diameter. They are sometimes found floating freely in waters inhabited by fresh-water sponges. As to the methods of germination, Mr. Potts' monograph may be referred to for the detail.

Most of the fresh-water sponges are characterized by a more or less vivid green color, which, however, is not universal, but, according to Mr. Potts, is closely dependent on the quantity or quality of the light received. A sponge which has germinated away from the light will be nearly white, gray or cream colored. If brought into full sunlight, it gradually becomes green, finally attaining a bright vegetable green. Some species, as for instance, *Meyenia leidyi*, are stated to never become green. On this account, *Meyenia leidyi* may be expected to grow in

* *Transactions*, Am. Soc. C. E., Vol. xv, p. 337.

† *Transactions*, Am. Soc. C. E., Vol. xxxv, p. 241.

‡ "Contribution Towards the Synopsis of the American Forms of Fresh-Water Sponges, with Descriptions of those Named by other Authors in all Parts of the World," by Edward Potts. *Proceedings*, Academy of Natural Sciences of Philadelphia for April-August, 1897.

Mr. Rafter. water conduits, where there is entire absence of light. This species is further characterized by a persistent habit through which the growths of successive seasons rise one above another, forming series of thin laminæ. In this way it sometimes builds up smooth rounded prominences of compact texture. The fact that it is sometimes found at considerable depths, may also be taken to indicate that this species may be found on the interiors of water conduits.

The fact that most of the sponges are naturally of a bright green color must, so far as present information goes, be taken to indicate that their natural habitat is in places sometimes subject to exposure to light, although many of the species undoubtedly prefer subdued light. Professor Lankester has shown the occasional occurrence of chlorophylloid green coloring matter in the tissue of animals, among others in fresh-water sponges, in *Meyenia fluviatalis*. A fact of this character tends strongly to show that the natural habitat is to be found in localities at any rate receiving some light, although it ought not to be overlooked that several species grow in places where the light is subdued. Without going into the subject extensively, the writer will for the present merely state his opinion that this is, with many species, largely a question of convenient attachment to a fixed support, rather than a necessary elementary condition. At any rate, the presence of the chlorophylloid coloring substance, in either plants or animals, implies exposure to light.*

The fresh-water sponges are widely distributed throughout the United States, Mr. Potts stating that he has examined *Spongilla fragilis* from at least 32 localities in 18 States; *Spongilla lacustris* from 26 localities in 16 States; *Meyenia fluviatalis* from 25 localities in 14 States; and *Tubella Pennsylvanica* from 18 localities in 11 States. That the fresh-water sponges are subject to considerable modifications under varying environments is indicated by the statement of Mr. Potts that hardly any two specimens were found exactly alike in their so-called typical features.

The place of the fresh-water sponges, in the animal kingdom, is found in the sub-kingdom, Protozoa. They are, therefore, allied to the Infusoria and the Rhizopods, the lowest forms of invertebrate animals.

The fresh-water polyzoa are a much higher form, being found in the sub-kingdom, Mollusca, and hence related to the Cephalopods, Gastropods, and other relatively advanced forms of the Molluscan sub-kingdom. They possess a complex alimentary system, with an œsophagus, stomach and intestinal tract. They also have a nervous system and well-developed muscles. Some species grow attached, while others are free-floating. Fixed forms are usually found immov-

* For reference to Professor Lankester's determinations of chlorophyl in minute animal forms, see Sacks' "Text Book of Botany," p. 767.

ably fastened by their ectocysts to the lower surface of submerged stones or floating timbers. In their fully developed state, the polyzoa live in colonies, a large number of individuals, each with its own digestive tube, tentacles, nerve ganglion, muscles, generative organs, etc., each reproducing itself either by gemmation or fixed statoblasts, while all continue organically united in a single group. Reproduction is either by budding or by statoblasts. When from buds, they grow from the side of an adult polypide. Statoblasts may be either fixed or free. The free statoblasts are the founders of new colonies, while buds merely increase the number of individuals in each colony. Mr. Rafter.

Generally, the polyzoa do not thrive in places subject to the direct action of sunlight. Their natural habitat seems to be in dark places. Undoubtedly, some of the species may exist in entire absence of light, although whether they thrive vigorously in such places is not definitely known. *Cristatella* is, however, an exception. According to Professor Alman, this polyzoon delights in exposure to the full influence of the sun, and may be seen basking upon the upper side of submerged stones, or creeping over the stems of aquatic plants in lakes and ponds.* Mr. Hyatt, however, points out that *Pectinatella* apparently prefers strong light in July and August, but cannot stand it at all in October. In the summer months it is found abundantly distributed in exposed shallow waters, but in the fall it disappears from such locations, and can then be discovered only in shaded places several feet below the surface. Colonies of *Pectinatella* frequently grow several feet in diameter.

Many species of the fresh-water polyzoa show considerable color, although not always a chlorophyllaceous green. *Paludicella* is, however, an olive green color. Its single filaments are from $\frac{1}{20}$ to $\frac{1}{30}$ in. in diameter. It is somewhat unlikely that with an olive green color predominant in most normal specimens, the natural habitat is found in absolute darkness. *Fredricella* may be cited as a species which apparently prefers nearly absolute darkness. According to Mr. Hyatt, this form is only found in the darkest places. *Plumatella* is another form apparently preferring light, and which is frequently found growing from the ends of water grasses, without any protection whatever from light and heat.

Aside from the influence of light on the development of color, there are two other facts which tend to show that the fresh-water polyzoa are not likely to grow naturally in water conduits. In searching for them along streams and about ponds, they are only occasionally found in rapidly moving water, their preference being apparently for very slowly moving or quiet water. Nor, have they usually been found in water of more than 3 or 4 ft. in depth, the

* For convenient English literature of the fresh-water polyzoa, refer to "Alman's Fresh-Water Polyzoa," published by the Ray Society, in 1856, and to Hyatt's Observations on the Polyzoa, in *Proceedings* of the Essex Institute, Salem, 1866–68.

Mr. Rafter. under side of a floating plank or timber being usually the best place to find them. It is somewhat doubtful, therefore, if they readily stand heavy water pressure, and, hence, so far as the present information goes, we should only accept well-attested statements as to their presence in pressure conduits.

Another difficulty is as to the method of attachment. With a perfectly smooth interior surface, there is apparently no way by which polyzoa can attach themselves. But if we assume a broken coating with ragged projections, the case becomes very simple. Floating filaments or germinating statoblasts would be easily caught, and large colonies ultimately developed. As regards fresh-water sponges, the conditions of attachment are somewhat similar, although the sponge has a power of secreting a gluey substance with which it attaches itself to stones in running water, and which might enable it to become attached in slowly moving water within a conduit. In the case of the 48-in. pipe examined by Mr. FitzGerald, it appears that there was a break in the coating at nearly every point where a sponge was attached. But whether this was due to the action of the sponge or was the original cause of attachment, is unknown. Reasoning from purely *a priori* considerations, however, it appears somewhat probable that breaks in the coating were the prime cause.

The foregoing very brief account of the fresh-water sponges and polyzoa, in connection with decrease of carrying capacity of water conduits, is sufficient to show that there are several difficulties to be surmounted before a safe theory of deterioration, which shall have universal application, can be successfully formulated. Nevertheless, the writer does not wish to be understood as saying that in some cases decrease in carrying capacity may not proceed from these causes, although, broadly, the evidence is yet too indefinite to form a safe general hypothesis. That is to say, the writer accepts the well-observed special cases, but is not satisfied that the growth of such forms is the universal explanation. The known preference of the polyzoa for slowly moving or quiet water, together with the development of color masses akin to chlorophyl in the fresh-water sponges and possibly, also, in some forms of polyzoa, militates somewhat against the view that they develop other than accidentally under the conditions of rapid movement and absolute darkness prevailing in water conduits.

As regards chlorophyl in these animal forms, it is not believed that it performs quite the same functions that it does in green plants. It is intended to go no further than to state that these animal forms develop a chlorophylloid substance which is undoubtedly dependent upon the quantity of light received. For such forms a water conduit does not appear to be the natural habitat.

Nevertheless, the fact of finding polyzoa attached to the interiors of water conduits, the same as fresh-water sponges, ought not to be

ignored. Very interesting questions are opened up by such a discovery, and some engineer-biologist, endowed with the scientific spirit and a fair stock of patience, has here an opportunity to elucidate a problem which is not only scientifically interesting, but which has very important commercial bearings. But those who have not pursued working biology to some considerable extent may well await, in this matter, the opinion of the qualified expert, because probably in the entire range of science there is no one place where either preconceived opinions, or those founded on insufficient data, are so likely to be modified by study and experience as in considering the different manifestations of life and the modifications to which it is subject, under varying environment. Realizing this truth, the writer, therefore, stands ready, on the presentation of acceptable evidence, to modify whatever in the nature of opinion is here expressed. Mr. Rafter.

GEORGE H. FENKELL, Jun. Am. Soc. C. E. (by letter).—There is little doubt that the authors have obtained for large wood and riveted pipe the most satisfactory set of experiments ever published. The generalized results obtained, however, for 1897 and 1899, show a considerable variation, and it is impossible to ascribe the cause of these discrepancies to the effect of leaks in the apparatus or to anything else connected with the manipulation of the instruments or the readings of the same. Some of the single observations are obviously in error, as will be seen later, but, on the whole, they show very careful work, and most of the discrepancies must be looked for, either in the effect of curvature on the flow, the tunnel, the Venturi meters, or in the reductions of the observations. Mr. Fenkell.

The writer will not attempt to discuss the first three reasons given, but will confine himself entirely to a criticism of the reductions, the results of which the authors have published in Tables Nos. 3 and 4.

A method of judging the relative value of experiments on the flow of water through pipes, and enabling the student to omit such observations as prove to be unreliable, having given the velocity and the loss of head, first proposed by Gardner S. Williams, M. Am. Soc. C. E., and since used by him and Clarence W. Hubbell, Jun. Am. Soc. C. E., on a wide class of work, is used by the writer.

If $v^2 = 2gh$, $h = \frac{v^2}{2g}$ = velocity head = H_v. If, for each given velocity, the velocity head is calculated and is plotted with the loss of head (H_f), or friction, a straight line through the origin should be the resultant, if the loss of head varies as the square of the velocity.

In Figs. 6 and 7, all observations in each series are shown in this manner. It is a well-known fact that the most satisfactory way to study the relations existing between a series of two quantities is by plotting on squared paper; and it is generally customary to average the results thus obtained (if a straight line) by means of a fine thread,

Mr. Fenkell. which can be moved about until the best average seems to have been found. The writer knows of no discussion on this subject, and it seems to have hardly been considered by hydraulic engineers. In this discussion the average line is found by first finding the center of gravity of an entire series, by dividing the sum of the ordinates and the sum of the abscissas by the number of observations. The series is then divided into two parts by the center of gravity. The center of gravity of each of these two parts is found and is plotted on Figs. 6, 7, 8 and 9 with three concentric circles. This method was first suggested to the writer by Mr. C. W. Hubbell. These two points, which are in line with the center of gravity of the entire series, determine the position of the average line, and only in rare instances, with ordinary experiments, would this line pass precisely through the origin.

It is probable, however, that all errors due to observation, personal error of the observer, calibration of instruments and leaks or air bubbles in the gauge connections, are nearly constant for all velocity heads, and hence for all velocities, and if the average line for a series of points is moved parallel to itself until it passes through the origin, it will then be the average line if all errors are eliminated from the observations.

It is also evident, after plotting all points, that a few are in error to such an extent that they had better be dropped entirely. Most of these are mentioned by the authors as being of a doubtful nature, and in Figs. 6 and 7 they are all marked "omit." In the experiments on the wood pipe, for 1897, Manometer 3–4, Observations 35 and 46 are so treated. In the experiments on the wood pipe, for 1899, Manometer 3-4, Observations 35, 36, 50 and 51 were dropped, while with Manometer 5–6 all are considered reliable. Observations 16, 15, 10, 29 and 28 are omitted from the experiments on the steel pipe, for 1897, and Observation 30 from the 1899 experiments. The lines deduced from the reliable points are drawn in Figs. 6 and 7, and their equations are as follows, H_v referring to velocity head and H_f to loss of head per thousand feet, or friction:

Six-Foot Wood Pipe.

Manometer 3-4, 1897 ..... $H_f = 2.657\ H_v + 0.0313$
" " 1899 ..... $H_f = 2.803\ H_v - 0.0188$
" 5-6, 1899 ..... $H_f = 2.834\ H_v + 0.0079$

Six-Foot Steel Pipe.

Manometer 1-2, 1897 ..... $H_f = 3.869\ H_v - 0.0079$
" " 1899 ..... $H_f = 3.850\ H_v + 0.0358$

These equations are the average lines as plotted, and the constant denotes the distance on the vertical axis that the line passes from the origin. If each one is moved parallel with itself until it passes through the origin, they then drop the constant and become:

Mr. Fenkell.

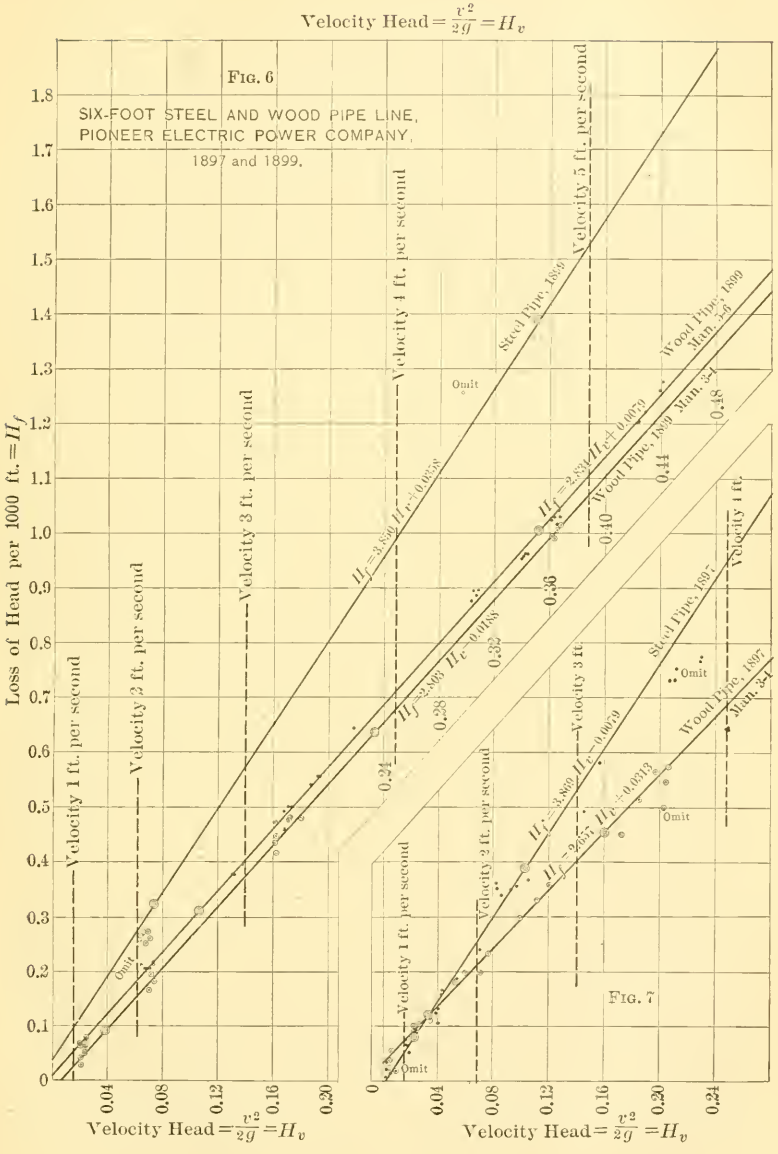


FIG. 6

SIX-FOOT STEEL AND WOOD PIPE LINE, PIONEER ELECTRIC POWER COMPANY, 1897 and 1899.

FIG. 7

Mr. Fenkell.

SIX-FOOT WOOD PIPE.

Manometer, 3-4, 1897.....$H_f = 2.657\ H_v$
" " 1899.....$H_f = 2.803\ H_v$
" 5-6, 1899.....$H_f = 2.834\ H_v$

SIX-FOOT STEEL PIPE.

Manometer, 1-2, 1897.....$H_f = 3.869\ H_v$
" " 1899.....$H_f = 3.850\ H_v$

These equations show the relation existing between the velocity head and the loss of head, and, consequently, with a given velocity, by multiplying its velocity head by the coefficient of H_v, as given in any of the foregoing equations, the result will be the loss of head per thousand feet for that particular pipe. Table No. 8 has been deduced from these equations, and shows the results as compared with those given in Tables Nos. 3 and 4, and the values of c, in the formula $v = c\sqrt{r\,s}$, as compared with those given by the authors.

It is evident that if the straight line passes through the origin, $H_v \propto H_f$. If $v = c\sqrt{r\,s}$, $c = \frac{v}{\sqrt{r\,s}}$. As $v^2 \propto H_f \propto H_v$, and as r is constant for each size, c will remain constant for all velocities in the same pipe, as shown in the table. This method of reducing the observations shows considerable decrease in the carrying capacity of the wood pipe, while that of the steel pipe is slightly increased. This increase is so small, however, that it should not be considered, as it represents a fineness hardly justified by the experiments. It does show, however, that the carrying capacity of the steel pipe, as nearly as can be calculated, was the same in 1899 as in 1897.

Fig. 8 represents the observations on the 38-in. steel conduit, at Rochester, made by Emil Kuichling, M. Am. Soc. C. E., in 1897,* and the writer believes them to be the best experiments on large riveted pipe ever made at low velocities. In Table No. 9 the loss per thousand feet and values of c are compared with those published by Mr. Kuichling. This table was deduced in the same manner as those for the 6-ft. wood and steel pipes.

Fig. 9 shows a few of the results on the East Jersey Water Company's 42-in. riveted pipe,† obtained by Clemens Herschel, M. Am. Soc. C. E. Although a large number of observations were made on this company's conduits, they were taken on so many different lengths, with but few series on the same pipe through a range of velocities, that only a part of them is capable of being worked up in this manner. Many of them show wide discrepancies; that which is shown being the best one published. Fig. 9 also shows, in like manner, the generalized results, taken from an average line, of the experiments

* "Annual Report, Executive Board," Rochester, 1897.

† "115 Experiments on the Carrying Capacity of Large Riveted Metal Conduits, up to 6 ft. per second of Velocity of Flow," by Clemens Herschel, M. Am. Soc. C. E.

Mr. Fenkell.

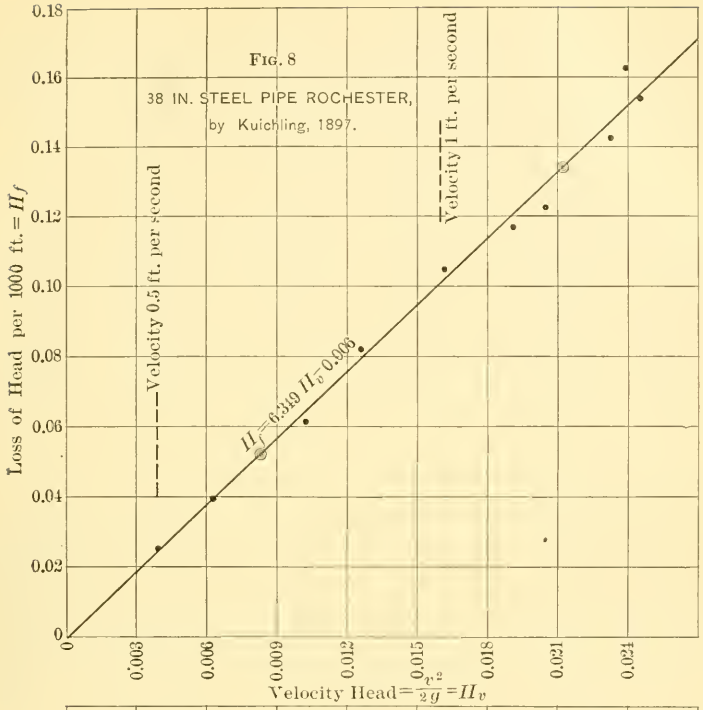


FIG. 8
38 IN. STEEL PIPE ROCHESTER,
by Kuichling, 1897.
Velocity 0.5 ft. per second
Velocity 1 ft. per second
$H_f = 6.349\ H_v - 0.008$
Loss of Head per 1000 ft. $= H_f$
Velocity Head $= \frac{v^2}{2g} = H_v$
0.18
0.16
0.14
0.12
0.10
0.08
0.06
0.04
0.02
0
0.003
0.006
0.009
0.012
0.015
0.018
0.021
0.024

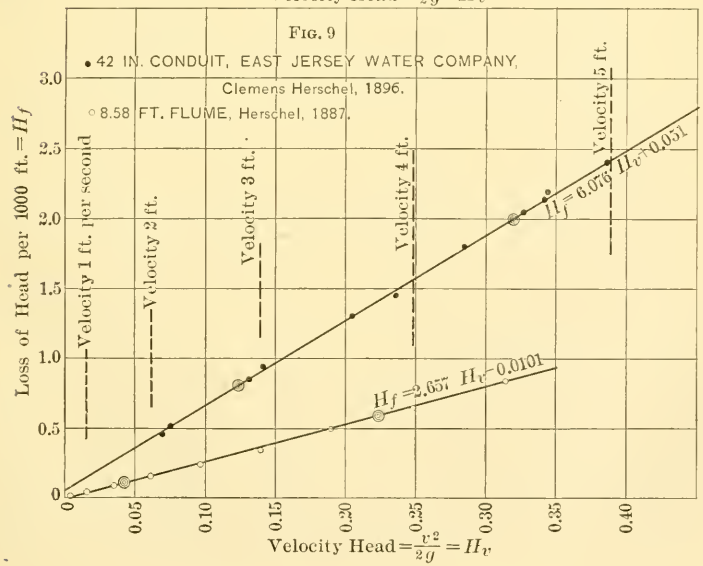


FIG. 9
• 42 IN. CONDUIT, EAST JERSEY WATER COMPANY,
Clemens Herschel, 1896.
○ 8.58 FT. FLUME, Herschel, 1887.
Velocity 1 ft. per second
Velocity 2 ft.
Velocity 3 ft.
Velocity 4 ft.
Velocity 5 ft.
$H_f = 6.076\ H_v - 0.051$
$H_f = 2.657\ H_v - 0.0101$
Loss of Head per 1000 ft. $= H_f$
Velocity Head $= \frac{v^2}{2g} = H_v$
3.0
2.5
2.0
1.5
1.0
0.5
0
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40

Mr. Fenkell. on an 8.58-ft. flume,* 152.88 ft. long, by Clemens Herschel, M. Am. Soc. C. E. As the single observations were not published, it is impossible to tell the amount of averaging necessary to produce the published results. A comparison of published results and those calculated by the writer on these pipes is shown in Table No. 11.

TABLE No. 8.

SIX-FOOT WOOD PIPE. GENERALIZED RESULTS.

	1897.					1899.					
	MANOMETER 3-4.					MANOMETER 3-4.		MANOMETER 5-6.			
Velocity.	Velocity Head. H_v.	Loss per 1000 ft. H_f.	Loss per 1000 ft. Table No. 4. H'.	c. From Column 3.	c. From Table No. 4.	Loss per 1000 ft. H_f.	c. From Column 7.	Loss per 1000 ft. H_f.	Loss per 1000 ft. Table No. 4. H'.	c. From Column 9.	c. From Table No. 4.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
1.0	0.0155	0.041	0.066	127.3	100	0.043	124.5	0.044	0.049	123.2	116.0
1.5	0.0349	0.093	0.123	127.3	110	0.098	124.5	0.099	0.106	123.2	118.7
2.0	0.0621	0.165	0.200	127.3	115	0.174	124.5	0.176	0.184	123.2	119.9
2.5	0.0971	0.258	0.292	127.3	119	0.272	124.5	0.275	0.284	123.2	120.8
3.0	0.1398	0.371	0.400	127.3	122	0.392	124.5	0.396	0.404	123.2	121.4
3.5	0.1902	0.505	0.527	127.3	124	0.533	124.5	0.539	0.548	123.2	121.7
4.0	0.2485	0.660	0.678	127.3	125	0.697	124.5	0.704	0.712	123.2	122.0
4.5	0.3144	.....	.....	.....	.....	0.881	124.5	0.891	0.898	123.2	122.2
5.0	0.3882	.....	.....	.....	.....	1.088	124.5	1.100	1.105	123.2	122.4
5.5	0.4697	.....	.....	.....	.....	1.317	124.5	1.331	1.335	123.2	122.5

SIX-FOOT STEEL PIPE. GENERALIZED RESULTS.

	1897.					1899.			
	MANOMETER 1-2.					MANOMETER 1-2.			
Velocity.	Velocity Head. H_v.	Loss per 1000 ft. H_f.	Loss per 1000 ft. H'.	c. From Column 15.	c. From Table No. 3.	Loss per 1000 ft. H_f.	Loss per 1000 ft. From Table. No. 3.	c. From Column 19.	c. From Table No. 3.
(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)
1.0	0.0155	0.060	0.055	105.5	110	0.060	0.100	105.6	81.6
1.5	0.0349	0.135	0.121	105.5	111	0.134	0.177	105.6	92.0
2.0	0.0621	0.240	0.220	105.5	110	0.239	0.277	105.6	98.0
2.5	0.0971	0.376	0.356	105.5	108	0.374	0.405	105.6	101.3
3.0	0.1398	0.541	0.510	105.5	108	0.538	0.570	105.6	102.4
3.5	0.1902	0.736	0.673	105.5	110	0.732	0.765	105.6	103.2
4.0	0.2485	0.961	0.863	105.5	111	0.957	0.987	105.6	103.8
4.5	0.3144	.....	.....	.....	.....	1.210	1.237	105.6	104.3
5.0	0.3882	.....	.....	.....	.....	1.494	1.516	105.6	104.7
5.5	0.4697	.....	.....	.....	.....	1.808	1.824	105.6	105.0

* *Transactions*, Am. Soc. C. E., Vol. xvii, 1887, and "115 Experiments," by Clemens Herschel.

Mr. Fenkell.

TABLE No. 9.—Rochester 38-Inch Steel Pipe, 1897.

Emil Kuichling, M. Am. Soc. C. E.

Velocity.	Velocity Head. H_v.	Loss per 1 000 ft. H_f.	Loss per 1 000 ft. Annual Report.	c. From Column 3.	c. From Annual Report.
(1)	(2)	(3)	(4)	(5)	(6)
1.23897	0.02383	0.1513	0.1629	113.2	109.07
1.25425	0.02442	0.1550	0.1545	113.2	113.41
1.22286	0.02322	0.1474	0.1418	113.2	115.41
1.15065	0.02057	0.1305	0.1229	113.2	116.63
1.10751	0.01907	0.1211	0.1172	113.2	114.98
1.02060	0.01618	0.1027	0.1048	113.2	112.03
0.89790	0.01255	0.0797	0.0819	113.2	111.53
0.81413	0.01029	0.0653	0.0615	113.2	116.49
0.63742	0.00630	0.0400	0.0397	113.2	113.66
0.50525	0.00396	0.0251	0.0254	113.2	112.58

TABLE No. 10.—42-In. Conduit, East Jersey Water Company, 1896.

Clemens Herschel, M. Am. Soc. C. E.

No. of Observation.	Velocity.	Weight.	Velocity Head. H_v.	Loss per 1 000 ft. H_f.	Loss per 1 000 ft. "115 Exp."	c. From Column 5.
(1)	(2)	(3)	(4)	(5)	(6)	(7)
259.....	2.21	*B.*	0.0758	0.461	0.52	110.1
260.....	3.02	*B.*	0.1416	0.86	0.93	110.1
261.....	3.90	*B.*	0.2362	1.44	1.45	110.1
262.....	4.69	*C.*	0.3403	2.08	2.14	110.1
263.....	4.59	*B*	0.3271	1.99	2.04	110.1
264.....	4.70	*A.*	0.3415	2.08	2.18	110.1
265.....	4.29	*A.*	0.2858	1.74	1.80	110.1
266.....	3.63	*A.*	0.2045	1.24	1.31	110.1
267.....	2.91	*A.*	0.1315	0.80	0.85	110.1
268.....	2.10	*A.*	0.0686	0.42	0.47	110.1
270.....	4.99	*A.*	0.3868	2.35	2.40	110.1

TABLE No. 11.—8.58-Ft. Flume at Holyoke, 1887.

Clemens Herschel, M. Am. Soc. C. E.

No. of Observation.	Velocity.	Velocity Head. H_v.	Loss per 1 000 ft. H_f.	Loss per 1 000 ft. "115 Exp."	c. From Column 4.	c. From "115 Exp."
(1)	(2)	(3)	(4)	(5)	(6)	(7)
501.....	0.5	0.0039	0.0104	0.0078	106.1	126.5
502.....	1.0	0.0155	0.0412	0.0320	106.1	116.6
503.....	1.5	0.0349	0.0927	0.0822	106.1	112.7
504.....	2.0	0.0621	0.1650	0.1532	106.1	110.3
505.....	2.5	0.0971	0.2580	0.2421	106.1	108.8
506.....	3.0	0.1398	0.3714	0.3520	106.1	107.7
507.....	3.5	0.1902	0.5054	0.4902	106.1	106.9
508.....	4.0	0.2485	0.6603	0.6520	106.1	106.2
509.....	4.5	0.3144	0.8354	0.8350	106.1	105.6

Mr. Fenkell. In order to test a set of experiments by this method, it is necessary to have a considerable range of velocities on the same length of pipe, and the first series by the authors was the first set so taken on large stave pipe. Figs. 6 and 7 show the best sets of experiments on large riveted pipe, to date, which can be tested by this method. It is to be regretted that the experiments on the Astoria wooden and riveted pipe, by Arthur L. Adams,* M. Am. Soc. C. E., are not capable of such comparison.

Mr. Henny. D. C. HENNY, M. Am. Soc. C. E. (by letter).—In his discussion of the first series of experiments the writer touched upon some elementary factors which in the first paper had, in his judgment, failed to receive the attention they deserved. Being cognizant of the fact that the first experiments were to be supplemented and extended, he hoped that an opportunity would be sought to remove more fully the doubt which seemed justified regarding some of these points. In this respect, the writer confesses to some disappointment, the more severe as he realizes the scarcity of available information and the perhaps unduly enhanced value which is likely to be accorded to individual experiments.

Diameter.—In the present paper the authors have continued to assume the interior diameter of the stave pipe at 72½ ins. No mention is made of any check, by actual measurement, upon this assumption, which, for reasons stated, the writer considered as probably incorrect. He can now add that, last January, he made a hasty examination of the wooden pipe at the point where it crosses the trestle, at Station 276, at which point Manometer No. 3 was located. He found that while the assumed interior diameter, 72½ ins., corresponds with the projection of thread at each lug of 3¾ ins., the actual projection exceeded this in every case. On a few of the bolts he measured a projection of 8 ins., made possible only by the placing of dozens of washers or fillers under the nuts. Whether the bolts themselves were longer than stated by Mr. Goldmark in his paper† on the Pioneer Power Plant the writer had no means at hand for determining. The exposed portion of the pipe had the appearance of having been re-cinched, after completion, to stop leakage, it being observed incidentally as an interesting fact, that many of the pressed-steel lugs had become badly deformed, the side walls having generally bent inward until in some cases they touched at the top.

The possibility of such reduction in diameter emphasizes the uncertainty, well known to wooden-pipe builders, of basing an estimate of the probable diameter of a stave pipe upon the original width of the individual staves, as appears to have been done by the authors.

It is not contended that the apparent reduction of diameter here

* *Transactions*, Am. Soc. C. E., Vol. xxxvi, p. 1.

† *Transactions*, Am. Soc. C. E., Vol. xxxviii, p. 270.

observed renders a similar excessive reduction likely where the pipe is buried. The probabilities are the other way, yet it leaves it pertinent to ask why the authors have omitted to present measurements of the outer circumference of the pipe, readily obtainable at the various points where the pipe passes over bridges and trestles. Even if such measurements could not be accepted as true averages they would have defined more clearly the relative importance of the possible error due to this cause. The writer believes that at the point examined by him the pipe may well have been less than 71 ins. in diameter, and have had an area close to 5% less than that assumed. Mr. Henny.

It was contended by the authors in their first paper* that there was no need of such measurements: First, because similar uncertainty as to diameter would exist in other stave pipe, which the writer cannot admit, because experience makes it possible, in the construction of stave pipe, to approximate closely to the desired diameters; and second, because the possible error attributed to this cause would be insignificant for the purpose of explaining the disparity in results of the Ogden and previous experiments, which is also objected to as imposing an illogical and irrelevant limitation upon the accuracy of the experiments.

Presence of Air.—The writer has failed to find any information as to the air which had accumulated at summits where it could be blown off. Check valves to be depressed with a bar afforded the only means for releasing air, and these did not occur at all summits, judging from the profile† presented by Mr. Goldmark. In this respect the long section of stave pipe is not as thoroughly protected as the shorter section with the tunnel relief shaft at its upper end. Air at summits, where it cannot be blown off, does not, in the writer's opinion, necessarily reveal its presence through any irregularity in the results of experiments.

Presence of Sediment.—It was considered highly improbable by Mr. Goldmark that sediment could have affected seriously the first experiments on the short section of stave pipe, because the pipe had been in use only a few months previous to the time the experiments were made. This argument does not hold good in the present instance, two years having elapsed. The importance of this matter has become forcibly impressed upon the writer's mind by a recent occurrence which is of interest in this connection.

The light wooden trestle carrying the 52-in. inverted stave-pipe siphon, on the line of the Santa Ana Canal‡ across Deep Cañon, near Redlands, Cal., was originally designed for a load equal to the weight of the pipe and water immediately over it. No account was taken, so far as the writer is aware, of the additional load due to thrust from

* *Transactions*, Am. Soc. C. E., Vol. xl, p. 557.

† *Transactions*, Am. Soc. C. E., Vol. xxxviii, p. 255.

‡ *Transactions*, Am. Soc. C. E., Vol. xxxiii, Plate xvii.

Mr. Henny. vertical curvature, amounting, as was shown in his discussion of the paper mentioned, to over 40% of the load figured on. The trestle settled considerably at the time of filling the pipe, and in the course of years showed further dangerous signs of weakness, until, a short time ago, a portion of it collapsed, carrying a part of the pipe down with it. It then became apparent that, for about 60 ft. in length, the pipe had gradually filled with fine sand, which had become indurated, and which had left only a small passage for the water in the lower reaches of the siphon. The sediment had added to the load, and had probably contributed materially to the disastrous result. So far as this incident concerns the present discussion and the possibility of the formation of sediment in pipe lines generally, it should be understood that during the last three or four years, only a very small flow of water had passed through the pipe, which had thus been converted into a long settling basin; and that the sand-box at the upper end of the canal may not have been effective or properly operated. Yet, considering all circumstances, the probability of a serious amount of sediment forming in this siphon did not seem great.

The Ogden pipe takes water directly from a similar mountain stream; its intake is believed to be unprovided with any sand-trap or settling basin, and the writer understands that the small percentage of the total available power ordinarily utilized calls for only a low velocity in the pipe.

Unless definite information can be presented, tending to prove the pipe to have had a clear section, it is necessary to fall back upon a comparative study of results, and with this end in view Table No. 12 has been deduced.

TABLE No. 12.—Experiments on Six-Foot Stave Pipe.

Group.	Velocity, in feet per second.	Value of c in the Chezy Formula.		
		Short Section.		Long Section.
		1897.	1899.	1899.
		Deduced from average curve.	Calculated from average of observations for each group	
(1)	(2)	(3)	(4)	(5)
J	1.175	104	155	117
N	1.244	106	137	118
L	2.126	116	116	121
I	2.144	116	118	119
K	3.239	123	127	121
M	3.324	123	124	121
T	4.845	.....	125	123

Table No. 12 gives the values of c for the velocities indicated in Column 2. Column 3 gives these values as deduced from the average curve deduced from the 1897 experiments on the short section of stave pipe. Column 4 gives the values found on the same section of pipe in 1899, and Column 5 gives the values found on the long section of pipe in 1899. Mr. Henny.

The grave disparity which may be observed in the results for low velocities is sufficient to cause their rejection. For velocities above above 1.5 ft. per second the results agree in a satisfactory manner.

As regards the presence of sediment, the comparison gives undoubted weight to the assumption that if no sediment were present in the short section of pipe in 1897, there probably was none in 1899. From the close agreement of the experiments on the short and long section (Columns 4 and 5), the further deduction may be made that the long section must also have been practically free from sediment. The writer believes, however, that this chain of reasoning is not sufficiently strong to depend on for conclusions, where the physical facts in the case, so far as understood by the writer, rather favor the inference that some unknown amount of sediment may have been present in the pipe. Blow-off gates, as argued before, would, upon their first discharge, have afforded the means of establishing the presence or absence of sediment at least at or near the points where attached. If they have been utilized for this purpose the writer has failed to find any mention of it by the authors.

Curvature.—Experiments to determine the effect of sweeping curves upon the loss of head in pipe lines are yet to be undertaken. Measuring the loss of head due to short and sharp curves is not believed to furnish a safe guide in this respect. Short elbows alternating with long tangents in a pipe line are likely to have only a local effect upon the motion of the particles. Skin friction in the straight reaches will tend to oppose the disturbance and will favor, and probably reproduce, approximately parallel and rectilinear motion. With long curves, the centrifugal force, while smaller in amount, has longer time to act and to establish a *régime* of its own for each particular curve, which, whatever it may be, whether a generally rotating motion, or arbitrary eddying, is sure to increase the frictional loss. With comparatively short tangents intervening, the new *régime*, after being established, will extend into the next curve, and be either confirmed or destroyed to make place for another general scheme of movement, and every change of motion involves additional loss.

With a pipe line, such as was available to the authors, having hardly any true tangents, it was impracticable to study the effect of curvature independently. Moreover, it must be admitted that most wooden pipe lines have a considerable percentage of easy curvature. Being familiar with a large number of stave pipe lines now in use, the

Mr. Henny. writer will say, however, that he knows of none where the percentage of true tangents, especially if expressed in diameters, is so small as would appear to be the case with the Ogden pipe line, judging from the plan and profile shown in Mr. Goldmark's paper. And especially is this true if the Ogden pipe line be compared in this respect with stave pipe lines elsewhere, experiments whereon have been recorded. Speculation on this subject may be fruitless, yet it emphasizes the necessity of entering this field analytically before hydraulicians can hope to make material progress.

General Remarks.—The writer wishes to reiterate, that possible presence of air and sediment, as well as probable deficiency in diameter, if causing error at all, have produced cumulative results in increasing the value of c; while an unusually high percentage of length of curves tends in the same direction. Whether the combined effect may be deemed sufficient to bring these experiments in line with those previously undertaken with smooth surfaced conduits of various classes, or whether a serious doubt should be entertained as to the accuracy of the Kutter formula as applied to smooth-bore pipe, can hardly be settled satisfactorily until additional light be thrown upon this complex subject. In the absence of additional and convincing information to the contrary, the writer inclines to the former proposition. Moreover, the new experiments on the stave pipe, taken by themselves, constitute a forcible confirmation of the accuracy of the Kutter formula so far as it accounts for change in velocity.

The experiments on the steel pipe strengthen the belief that the Kutter formula offers no advantage over the simple Chezy formula.

On the important subject of effect of time on the carrying capacity, the experiments shed some valuable light. Table No. 4 is, in this respect, not as conclusive as Table No. 12, since the latter permits of a comparison of the experiments of 1897 and 1899 on the same section of stave pipe. The value of c shows no diminution during the two years' interval. With the steel pipe, the diminution in carrying capacity is very marked, considering the short time the pipe has been in use, and while the earlier experiments indicate a superiority in the carrying capacity of stave pipe over steel pipe, of about 12% for $3\frac{1}{2}$ ft. velocity, this difference appears to have increased, in the course of two years, to 20 per cent.

Mr. Sherman. Charles W. Sherman, Jun. Am. Soc. C. E. (by letter).—Mr. Rafter's discussion on the effect of animal and vegetable life on the discharging capacity of a pipe is very interesting, but needs to be supplemented by records of actual observation of the presence or absence of such growths. The writer submits these few lines in the hope of adding his mite to the meager data on this subject.

Late in the year 1894, a 36-in. force main was laid from the Chestnut Hill Pumping Station to the Fisher Hill Reservoir of the Boston

Water-Works, the distance being something over a mile. Early in the following year, during a test of the new Leavitt pumping engine, observations of the loss of head were taken at three points in a length of about 5 000 ft., from which it was found that the coefficient c of the Chezy formula was 136, v being 4.7 ft. per second, or about what would be expected for a new cast-iron pipe of this size. A year later, or early in 1896, an extensive series of experiments on the friction loss in this pipe was made by the writer, under the direction of Desmond FitzGerald, Past-President, Am. Soc. C. E., with velocities ranging from 1.1 to 4.5 ft. per second. It was supposed that the pipe was in about the same condition, but the results showed a coefficient of about 113, a great loss within a single year. This result was so far from that expected that the pipe was partly drained, and entered at the upper end. Only a small amount of tuberculation was found, but the whole interior surface was covered with a slimy substance which proved upon examination to consist almost wholly of the polyzoon *Fredericella*. In 1897, another experiment, in which v was 3.2 ft. per second, showed c to be about 114. It thus appears that there was a great change in the capacity of the pipe in the first year after it was laid, and practically none in the year following. Mr. Sherman.

Late in 1897, this pipe was cut into, for the purpose of making connections near the pumping station, and an examination of the interior surface at this point showed that practically the same conditions obtained. If anything, the organic growth was somewhat thicker, as was to be expected, this being nearer the source of the food supply.

It seems to the writer that, with New England surface waters, which contain more or less organic matter, such as algæ, to furnish food for them, growths of polyzoa on the interior of the pipes are to be expected, and the result will be a large diminution of the capacity of the pipes within the first year. After that there will be a further gradual diminution of capacity due to the slow increase of tuberculation. With ground-waters or filtered waters, which have not been exposed to the light, such growths will probably not occur, as food for the polyzoa will be lacking. The experience in Brookline, which has a ground-water supply, and where such growths have not been observed, seems to bear out this opinion.

The writer is very glad that Professor Williams has drawn attention to the necessity for some criterion other than the reputation of the experimenter, and the apparent accuracy as indicated by the method of least squares, to give some indication of the reliability of experiments. The method used by Professor Williams and Mr. Fenkell is, doubtless, of great value, as indicating what experiments of any series should be rejected. It involves, however, both a needless assumption and an unnecessary amount of work in its application. If the

Mr. Sherman. logarithms of the velocities be plotted as abscissas, with the logarithms of losses of head as ordinates, a straight line can then be drawn to represent the plotted points, without the laborious computation of, or even the necessity of taking from a diagram, the values of $\frac{v^2}{2g}$, or the "velocity head." By the use of logarithmic cross-section paper, this plotting can be done with great rapidity.

An especial advantage of this method is that it does not require the assumption that the loss of head varies as the square of the velocity: It is assumed only that this loss varies as some unknown power of the velocity. The criterion, that is, the comparison of plotted points with the line representing the series, is similar to that proposed by Professor Williams.

This application of the logarithmic diagram is not new. As far as the writer knows, it was first introduced to this Society by Desmond FitzGerald, M. Am. Soc. C. E., in his paper on "Flow of Water in 48-In. Pipes."* Mr. FitzGerald refers to a paper before the Royal Society for the origin of this method. To be sure, it is not there noted particularly that this furnishes a criterion for the acceptance or rejection of particular experiments in a series, as the method is used in the derivation of an exponential formula. The advantage of the logarithmic diagram over that obtained by plotting "velocity heads" can, perhaps, be expressed no better than in Mr. FitzGerald's words, "Resistances do not vary exactly as the square of the velocity, however much the text books may insist to the contrary."

Mr. FitzGerald. DESMOND FITZGERALD, Past-President, Am. Soc. C. E. (by letter).—The writer has yet to see a structure for conveying water which will have the same capacity after it has been some time in service as it had when new. There may be some material which will not change its character, and upon which animal or vegetable life will not grow, but the writer has not met with such material in his experience. It is, however, extremely dangerous to generalize in engineering matters. There are hardly two waters exactly alike in chemical composition, and serious mistakes may be made by applying the results of experience from a soft New England water to a hard, lime-carrying western water. The writer has often been informed that certain structures for conveying water have as large capacity at the end of several years as they had when new, but closer examination has shown that the data were not sufficiently accurate to settle the question.

The experiments made on large pipes, under the direction of the writer, have already been referred to by Mr. Charles W. Sherman, so that it is unnecessary to say much more in regard to them, except to add that the water was measured carefully by weir, that the losses of head were ascertained from observations on extremely accurate mer-

* *Transactions*, Am. Soc. C. E., Vol. xxxv, p. 241.

cury gauges, and that proper corrections were made in working out the results. Mr. FitzGerald.

The writer has also made experiments on the flow of water through cement-lined pipes, and has been very much surprised to find that there were large frictional losses, almost as great as those arising from old cast-iron pipes, and the only way he can account for this is that even cement-lined pipes do not remain perfectly clean for an indefinite time.

For many years the writer has had charge of the Cochituate and Sudbury Aqueducts and of the determination of their capacities, and he has been very much impressed with the effect on the flow of the water due to slimy growths on the surface of the brickwork. In the case of the Cochituate Aqueduct, if the deposits of slime and dirt are allowed to collect on the interior, they gradually give place to growths of Spongilla, and if these are allowed to accumulate, after a year or two, they become very hard, with long finger-like growths, which are serious obstructions to the flow. With changes of flow, this accumulated dirt on the walls of the aqueduct affects the quality of the water, so that, aside from the question of friction, there is also the important question of the effect on the quality.

In the case of the Sudbury Aqueduct, it has been found that, if the surface of the brick-work is not cleaned, in the course of a year it may accumulate sufficient slime to diminish the maximum capacity about 10 per cent. Some years ago the writer devised a method of cleaning, by a machine which worked under a pressure of water. The machine was carried on wheels with wooden tires, was provided with rotary brushes for cleaning the sides and bottom of the aqueduct, and worked very effectively. A movable shield in front held the water back so as to give the power. The machine was arranged to clean to a height of about 4 ft. only, and since it has become necessary to run larger quantities of water in the aqueduct, the machine has been taken out, and the whole length is now cleaned by hand. This is accomplished by a number of gangs of men, provided with rubber boots, stable and rattan brooms, and reflectors. It takes about nineteen men to clean a mile of aqueduct in a day.

Current meter observations have been taken on the flow in the Sudbury Aqueduct, and the results have been extremely accurate and satisfactory. On April 4th, 1899, with a flow of 74 000 000 galls. daily, the capacity of the aqueduct was only 92.58% of the tabulated capacity, and after cleaning on May 29th, at which time 95 000 000 galls. per day were flowing, the capacity was 103.95% of the tabulated capacity. It is possible that, where an aqueduct has been for a long time in use without cleaning, this decrease in carrying capacity may be found to be even greater than 10 per cent. The writer believes that the facts just cited are sufficient to demonstrate the commercial

Mr. FitzGerald. advantages, under some conditions, of keeping an aqueduct clean, as, for instance, when its full capacity is likely to be taxed.

Mr. Kuichling states that vegetable and animal growths are not often found on the bottoms of pipes, and this agrees with the observations of the writer, but the latter has seen the invert of the Sudbury Aqueduct almost completely covered with a thick growth of sponge.

Mr. Meem has stated that "This doubtless shows that organisms require more, as Mr. Whipple suggests, than that their food be brought to them, and they must also have a 'lodging place,' which is probably furnished in the steel and iron pipes by the commencement of rust." In connection with this matter, the writer desires to state, that the growths all around the interior of the new cast-iron pipes already alluded to, were in the nature of a greasy scum, and, that when this was washed off, the coating of the pipe underneath was as bright as when first laid.

The writer recently had occasion to lay a cast-iron pipe having a diameter of 61⅛ ins., and 1 800 ft. in length, as an inverted siphon. The reason for using this odd size was that it was found more economical to use the outside moulds already prepared for making a thick 5-ft. pipe, and to produce the thinner pipe, desired for the light pressures at the siphon, by increasing the size of the core. Every effort was made to lay this pipe with great care and to make very tight joints. Under careful test the leakage was found to average only 12 galls. per day for the whole length of the pipe, or 0.005 gall. per day per foot of joint. The interior portions of the joints were filled with Portland cement flush with the pipe and finished very smooth. The pipe was also given an extra painting. In this connection, it may be interesting to note that the average leakage per day per foot of joint from the 48-in. distribution pipes recently laid by the Metropolitan Water-Works was about 5 galls. per day per foot of joint.

The results of some experiments made to determine the carrying capacity of this siphon pipe gave a value of c, in the Chezy formula, of 149.6 for a velocity of 4.65 ft. per second and 154.3 for a velocity of 6.51 ft. per second. The writer believes that this coefficient, 154.3, is the highest ever recorded for cast-iron pipe, but there is no doubt in his mind that in the course of about twenty years this will be reduced by tuberculation to 100.

Messrs. Marx, Wing and Hoskins. Charles D. Marx, M. Am. Soc. C. E.; Charles B. Wing, Assoc. M. Am. Soc. C. E., and Leander M. Hoskins, Esq. (by letter).—It seems unnecessary to add to the discussion of the degree of reliability of the results, already given by the writers, in the paper on the experiments of 1897.

The questions raised by Mr. Henny have already been answered so far as the writers are able to answer them. It is possible that they underestimate the importance of exterior measurements of the pipe

Messrs. Marx, Wing and Hoskins.

at the trestles. If so, it is to be regretted that Mr. Henny did not make as complete a set of such measurements as seemed to him desirable. The writers are informed by Chief Engineer Bannister that many measurements of the interior diameter were made after the pipe was built, and that no appreciable departure from the estimated size was found. The writers certainly have no interest in maintaining that wood pipe offers greater resistance to flow than smooth iron pipe. Neither are they in possession of the data requisite for deciding the question. They do, however, feel warranted in the belief that the *a priori* estimates of the capacity of large stave pipes which have been published by some writers are far too great.

It is hard to believe that the elaborate discussions of Mr. Williams and Mr. Fenkell are intended as serious contributions to hydraulic literature. Certainly, no competent engineer is likely to regard seriously the proposition that many "supposedly reliable observations" are proved to be "nothing more than crude approximations," by the fact that they fail to show losses of head varying as the square of the velocity. Neither will conclusions which are based upon this "criterion," as to the change in capacity of the Ogden conduit, or as to the value of the results presented in the paper, be likely to meet with acceptance.

The supposition that the passage of water from the 9-ft. tunnel into the 6-ft. pipe generates a spiral motion sufficient to seriously affect the pressure readings at Station 4, 23 ft. from the tunnel, and to be noticeable even at Station 3, 2 733 ft. from the tunnel, does not appear to the writers to be intrinsically probable, or to be supported by the gauge readings at Stations 3 and 4.

As the discussion brings out somewhat diverse views upon the question of the change of capacity of the conduit between the dates of the two series of experiments, it may be well to refer again to this question.

In the discussion of the degree of reliability of the first series of results,* the fact was emphasized that it is impossible to determine very precise values of the coefficient c for low velocities, the reason being that a given error in the measurement of pressure causes an error in the estimated value of c which is far greater at low than at high velocities. While fully realizing this fact, it was thought best to extend the observations to as low velocities as practicable, not only because it was hoped that these observations might not be wholly without value in themselves, but because of the desirability of comparison with the results recorded by others, many of which, particularly upon stave pipes, have been made at low velocities. It appears, however, to be unsafe to base conclusions as to change of capacity upon the results obtained at velocities less than about 2 ft. per second.

* *Transactions*, Am. Soc. C. E., Vol. xl, p. 560.

Messrs. Marx, Wing and Hoskins.

As regards the steel pipe, the observations of 1899 probably show, with a somewhat higher degree of precision than the earlier series, the law of variation of loss of head with velocity. It is believed, however, that both series of results for the higher velocities may be accepted for the purpose of making a fairly correct estimate of the change in capacity of the pipe. The results shown in Table No. 3 indicate that the loss of head for a given velocity was about 13% greater in June, 1899, than in August, 1897.

Turning next to the wood pipe results, it must be remembered that the curve of loss of head for the experiments of 1899, shown on Plate VIII, is based wholly upon the observations on the long section of pipe above the tunnel, so that the two curves shown are not directly comparable for the purpose of estimating change of capacity. It cannot be assumed that the two portions of pipe should give identical values of the loss of head per thousand feet of length, especially when the different conditions of curvature are considered. The observations of 1899 on the short section of pipe are not sufficiently numerous to be made the basis of conclusions as to change of capacity. They do, however, show a fair agreement with the results of 1897, the only exceptions to this statement being Groups J and N, both taken at low velocities. The conclusions that appear to be warranted are:

(1) That the capacity of the short section of wood pipe changed only slightly, if at all, during the interval between the two series of experiments; and

(2) That in 1899 there was no important difference in capacity between the two portions experimented on.

The question, suggested by the writers and further referred to by Mr. Kuichling, as to whether the loss of head in the Venturi meters may not have increased, appears not to admit of a certain answer. Some light is perhaps thrown on the question by considering how the general results must be corrected if it be assumed that there has been an increased loss of head in the Venturi proper. This assumption would require the estimated velocities to be decreased, thus increasing the value found for the loss of head at any given velocity. This would imply a greater diminution of capacity of the steel pipe than above estimated, with a like result for the wood pipe. Attention has already been called (page 52) to the fact that if it be assumed that the observed increase in the loss of head between the first and third columns of the difference gauge is chargeable wholly to the part of the pipe between the throat and the down-stream section, this increase agrees well with that computed for the 6-ft. steel pipe. The most probable view appears to the writers to be that the Venturi meter coefficient remained practically constant.

The suggestion that the registration of the meters should be tested might well have been accompanied by a suggestion of specific

means of accomplishing it. No practicable method occurred to the writers. Messrs. Marx, Wing and Hoskins.

Regarding the desirability of examining the interior surface of the conduit, it should be said that the commercial operation of the plant does not admit of a shut-down of sufficient length of time to permit of draining the one mile of steel pipe, much less the five miles of wood pipe, unless damage to the plant should necessitate such draining.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

No. 880.

RECENT STADIA TOPOGRAPHIC SURVEYS: NOTES RELATING TO METHODS AND COST.

By WILLIAM B. LANDRETH, M. Am. Soc. C. E.

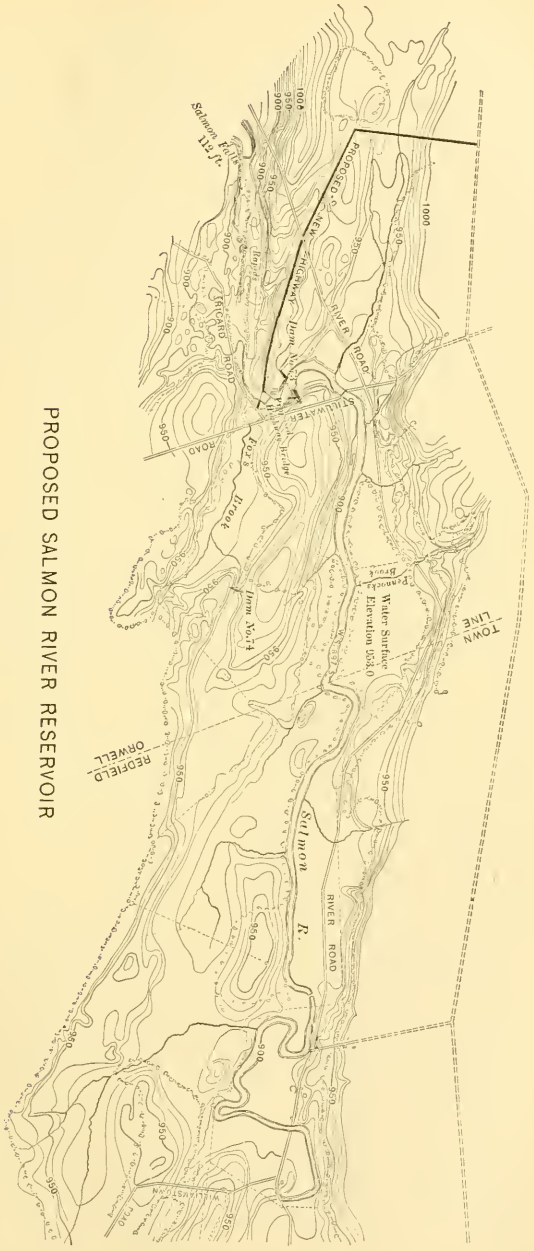
PRESENTED APRIL 4TH, 1900.

WITH DISCUSSION.

Surveys to determine the location of possible reservoirs and conduit lines for furnishing water for the summit level, or Rome level, of the proposed Deep Waterway on the Oswego-Mohawk-Hudson route were made under the direction of the Board of Engineers* on Deep Waterways, between August 1st, 1898, and June 1st, 1899. George W. Rafter was Chief of the Water Supply Divisions, with J. Y. McClintock, H. F. Northrup and the writer, Members, Am. Soc. C. E., as Assistant Engineers.

Mr. McClintock was in charge of the surveys on the feeder line from Carthage to Rome, N. Y. Mr. Northrup was in charge of the base-line surveys on the Salmon and Black Rivers, and of the topographic work on the latter stream near Carthage. The writer was in charge of the topographic work on the Salmon River, on the Black River beyond the limits of Mr. Northrup's survey, and on a portion of the feeder line, as a branch of Mr. McClintock's main party. Mr. B. E. Failing was

*Lieut.-Col. C. W. Raymond, Alfred Noble and George Y. Wisner, Members, Am. Soc. C. E.



PROPOSED SALMON RIVER RESERVOIR

FIG. 1.

transitman, and Mr. J. T. Parsons in charge of the office and map work, on the writer's party.

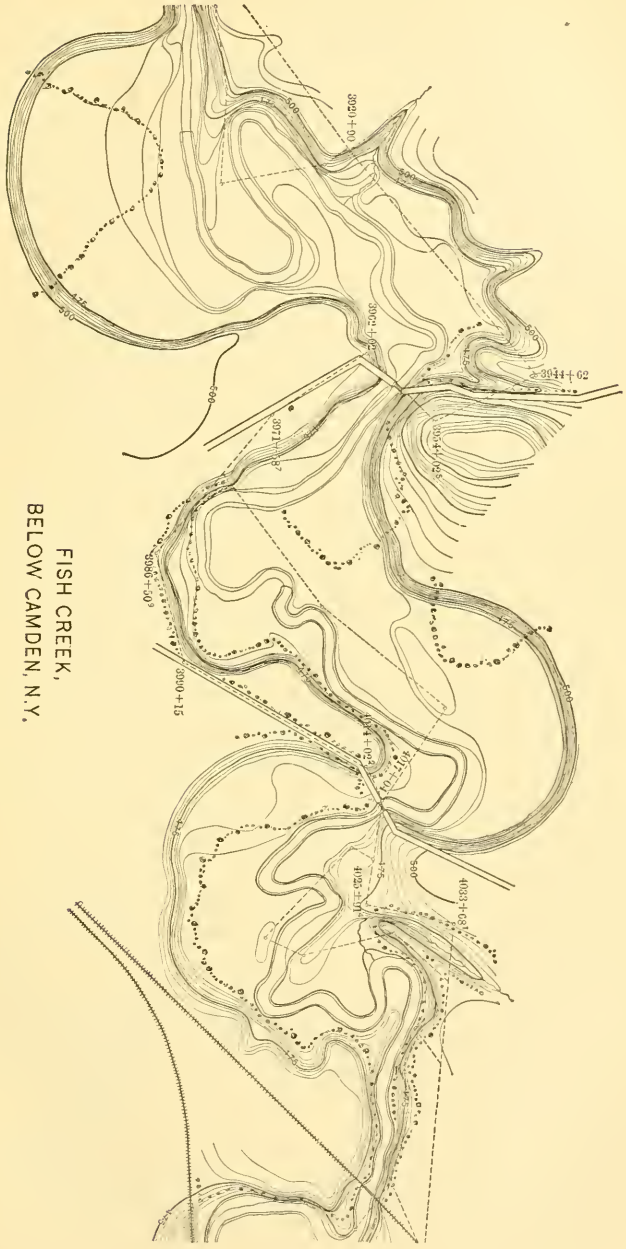
Party and Outfit.—The topographic party was composed of an assistant engineer in charge, a transitman, a recorder, three or more stadia rodmen, a draughtsman, a computer and two or more axmen. When running spirit levels, Mr. R. L. Burns, head stadia rodman, was detached from the main party for that purpose.

The outfit consisted of a transit fitted with non-adjustable stadia hairs, its horizontal limb reading to 20 seconds, and vertical arc to single minutes, one stadia rod for each rodman, a Y-level and rod; a large canvas umbrella, a tin megaphone, a 100-ft. steel tape, and the necessary office outfit of drawing table, instruments, etc. The whole outfit had been in use previously on the surveys of the main line of the Deep Waterway.

Field Work.—The method used in the topographic field work was that generally adopted on surveys where the distances are taken by stadia. The assistant engineer selected the base-line points to be occupied, and located the forward stadia points on the circuits, so as to cover the territory fully, and the party followed as closely as possible. Each rodman was given a particular class of objects to locate; one following the streams, another taking contour points, another the roads, buildings, woods, etc.

Each rodman worked independently, but when convenient all were kept on the same side of the transit, to facilitate the reading and plotting of the azimuths. An early trial was made of the method of placing all the rodmen on one azimuth, taking readings to each, and then moving all of them ahead to another azimuth; but it was soon abandoned as being productive of vexatious delays.

In working on streams of considerable size a circuit was run along one bank, and both slopes were worked from it, the party of rodmen being divided between them. Wooded gorges were worked from circuits on the top of each bluff, each slope being taken from the circuit on the opposite bluff. When the large streams were frozen, a circuit was run on the ice, and both banks were located therefrom. Crooked unfrozen streams were located from a circuit crossing them at the bends, with rodmen on each bank, the remainder of the party crossing from point to point in a boat. Long wooded slopes were taken from circuits along the top and foot thereof, and from short circuits con-



FISH CREEK,
BELOW CAMDEN, N.Y.

Fig. 2.

necting them, and so spaced as to cover the territory fully. Two corners of each building were generally located, the rodman measuring the other dimensions with his rod and giving the data to the recorder when passing him.

All streams, roads, buildings, outlines of woods, town and county lines and changes of slope were located. The contour interval on the surveys of the Salmon and Black River reservoirs was 10 ft., and on the Fish Creek feeder line 5 ft., with a 2-ft. interval on 2 square miles near the dam site on the Salmon River.

In rough ground the assistant engineer made field sketches on stiff-backed pads of cross-section paper, upon which governing points were located and numbered to correspond with the numbering of the shots locating them. It was found that the making of full sketches on all kinds of territory greatly retarded the progress of the field work without a corresponding gain in the rate of mapping.

Office Work.—At the close of each day's field work the field party reduced and checked the day's stadia readings and vertical-angle elevations, and the office force calculated the latitudes and departures of the stadia circuits. Two or more field books were in use at one time, the recorder changing books each day and leaving the book of the previous day for the use of the office force.

The mapping sheets were of heavy mounted white paper, 45 x 32 ins., with a working limit of 37 x 25 ins. The base, sub-base and stadia circuit lines were plotted carefully by latitudes and departures, care being taken to locate the working borders of each sheet so that the mapped surface would be evenly balanced thereon.

The sheets were lettered, inked and tinted to correspond with the same class of work done by the United States Geological Survey. The scale of the Salmon River and Black River maps was $\frac{1}{10000}$, and of the Fish Creek map $\frac{1}{5000}$.

Description of Territory.—The work on the Salmon River consisted of a survey of the valley of that stream above the Salmon River Falls, for a possible reservoir site. For a mile above the falls the gorge of the river is from 400 to 500 ft. wide with steep banks from 75 to 100 ft. high.

At the head of the gorge the valley widens out, forming flats from 1 to 1½ miles wide, which extend about 10 miles up stream.

The river flats are 900 to 910 ft. above tide water, the survey cover-

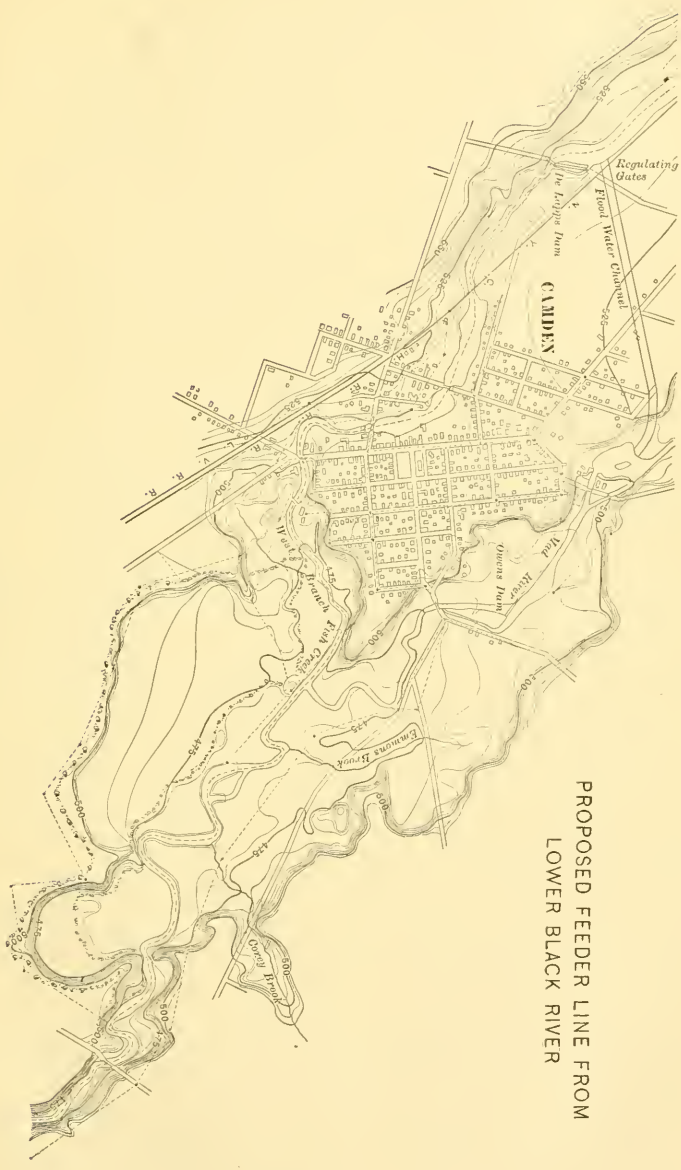

Regulating Gates
Flood Water Channel
De Lapps Dam
CAMDEN
West Branch
Fish Creek
Owens Dam
Wad River
Emmons Brook
Corey Brook
PROPOSED FEEDER LINE FROM
LOWER BLACK RIVER

FIG. 3.

ing the country up to the 970-ft. contour. About 65% of the area surveyed is covered with small second-growth timber and swamps. The north slope of the valley is an unbroken range of hills, but the south slope is broken, necessitating the careful location of, and spirit leveling over, several long dike sites. A range of partly wooded hills, from 50 to 75 ft. high runs lengthwise of the valley on the south side of the river, for a distance of 5 miles, with dense swamps between it and the south side. The village of Redfield, having a population of about 500, came within the limits of the survey.

The survey of the Fish Creek Valley extended from 2 miles above the village of Williamstown to 2 miles below the railroad station at Taberg, a distance by rail of 21 miles. The survey covered the flat portion of the valley and the side slopes and tributary streams to an elevation of 75 ft. above the stream. The ground covered was mostly grazing and farm lands, 40% of which was timbered. Below the village of Camden the stream runs in a crooked gorge from 200 to 600 ft. wide, having steep wooded walls from 75 to 150 ft. high, cut from an open plain. The villages of Williamstown, West Camden, Camden and McConnellsville lie within the area covered by the survey.

The survey for a reservoir site on the Black River covered the valley of that stream between the villages of Carthage and Lyons Falls, up to the 790-ft. contour. At Carthage the sides of the valley approach to within 500 ft. of each other, forming a natural dam site, with rock at a reasonable depth. Above Carthage the valley widens out, until at Castorland, 10 miles up stream, it is 8 miles wide. Opposite the village of Lowville, 19 miles from Carthage, the valley begins to narrow up, and at Lyons Falls, 40 miles from Carthage, the sides close together, forming falls 65 ft. high. Three large streams enter the valley from the east, forming long tributary valleys.

The river flats lie mostly between the 730 and 740-ft. contours; the river being canalized between Carthage and Lyons Falls, with dams and locks at Otter Creek and Bushee's Landing.

About 25% of the area covered by the survey is wooded, and the east side is partly covered with granite boulders, ranging in size from that of a barrel to that of an average city block.

An area of 15 sq. miles above the 790-ft. contour was surveyed for a relocation of the Black River Division of the New York Central and Hudson River Railroad, between the villages of Carthage and Lowville.

Accuracy of the Surveys.—As the use to which the finished maps were to be put did not call for great accuracy in the surveys, it was not sought. The stadia circuits checked within 15 ft. in latitudes and departures, and within 0.50 ft. in elevation, with a permissible limit of 1 ft. in the latter. The error on closure in the stadia circuits was from 1 in 1 200 to 1 in 1 400, their length varying from ½ mile to 8 miles. A 30-mile circuit of spirit levels closed within 0.09 ft., being within the limit of error allowed on the United States Geological Survey.

Rate of Progress and Cost.—Table No. 1 gives the rate, area covered, cost per square mile, and other data regarding the entire work.

TABLE No. 1.—Rate, Cost and Other Data of the Three Surveys.

Survey.	Dates.	Number of days worked.	Set ups.	Shots.	Miles in stadia circuit.	Old base line occupied. Miles.	New base line. Miles.	Spirit levels, in miles.	Area, in square miles.	Square miles per day.	Cost.			
											Field work, per square mile.	Map work, per square mile.	Total, per square mile.	Total.
Salmon River.	1898, Oct. 22d, Dec. 9th.	47	771	3 838	53.97	25.85	.....	1.92	15	0.32	$ 66.00	$ 14.00	$ 80.00	$ 1 200.00
Fish Creek.	1898, Dec. 13th, 1899, Feb. 3d.	44	451	11 776	56.13	24.87	.....	.....	19	0.45	54.00	25.00	79.00	1 500.00
Black River.	1899, Apr. 4th, May 27th.	47	600	11 166	138.48	34.40	17.80	56.66	85	1.81	16.50	7.00	23.00	2 000.00

The cost includes the salaries, maintenance, traveling expenses and supplies for the entire party, from the time of leaving the main office until the completion of the finished maps.

The cost of the base-line surveys for the Salmon River and Fish Creek work, and for one-third of the Black River, is not included in the cost as given. The area given in the table is that covered by the writer's party. The variation in the cost and rate per square mile on the several surveys is probably caused by the different conditions of weather, foliage and territory covered by them, and the increase for mapping on the Fish Creek survey is traceable to the use of a small contour interval and a large map scale.

DISCUSSION.

Mr. Low. EMILE LOW, M. Am. Soc. C. E. (by letter).—This paper recalls to the writer two topographical surveys made by him several years ago. One, a survey of Prince's Flats and vicinity, Wise County, Va., and the other, a portion of the property of the Mathieson Alkali Works, at Saltville, in Smyth and Washington Counties, Va.

Prince's Flats is now called Norton, and is the junction of the Norfolk and Western and Louisville and Nashville Railroads. It comprises a stretch of land between the Goest's and Powell Rivers, which streams rise to the north of the town, flowing southward almost parallel to one another only a few miles apart, and at Norton turn, the former flowing to the east, the latter to the west.

At the time of the survey, part of Prince's Flats consisted of a little cleared land, the remainder and the adjoining country being a virgin forest and a howling wilderness. The Norfolk and Western Railroad Company had obtained an option on the land, which had been selected for a town site and junction point. Before acquiring the land the company desired to have a topographical survey made, and this duty was assigned to the writer. The option had only a short time to run, and it became imperative to make the survey and map in the shortest possible time. The survey was made in one month by a small party organized for this purpose and consisting of an assistant engineer, transitman, chainmen, back flag and axemen.

The area to be surveyed was about 2 miles long and nearly $\frac{1}{2}$ mile wide, and consisted of several tracts of land, on one of which the boundary line had been run out by a local land surveyor. The territory was quite rugged, the difference in elevation between the lowest and highest points being about 800 ft. Many of the slopes ranged from 30 to 45 degrees.

A wagon road ran through the property from one end to the other, cutting it into two nearly equal parts, and it was decided to use this for the location of a base line. A transit line, marked by stakes at 100-ft. intervals, was run along the road, and from this base line, especially on the north side, spur lines were run up the various water courses and also along the tops of ridges and crests. On the south side additional base lines were run, generally parallel with the main base. The boundary lines were also re-run, and on all lines stakes were placed at 100-ft. intervals. Nearly all the lines were carefully levelled over with a wye level, and elevations of all stations were taken, as well as of intermediate points, where necessary. A few of the auxiliary lines were run by a second party, which happened to be on the ground at the time, making surveys for the proposed railroad.

After the levels had been run, the topography was taken. This

Mr. Low.

included all the natural as well as the few artificial features which existed, also the contours, which were taken at 5-ft. intervals.

The contours were taken by means of a hand or Locke level, the actual elevation ending in 0, and 5 ft. being ascertained on the ground and the distances from the center line noted. All measurements and observations were taken on lines at right angles to the base line. Some of the cleared areas were surveyed by the stadia method, in much the same manner as outlined in the paper.

The work of running the base lines, taking the levels of the same and the topography, so-called, was all done by one and the same party, each part of the work being taken up in regular sequence. The plotting was not done in the field, but at a neighboring farm house, on an improvised drawing table, after the base lines had been run, and while the levelling and topographical work was in progress.

The methods used in this survey gave very close results, it being especially desired that the location of the contour lines should be quite accurate, it being the intention to utilize the survey in laying out the town site, later on. A less accurate survey would have necessitated a second survey as a basis for laying out the streets. The streets were laid out several years after the survey was made, and it may be of interest to note, that very few, if any, changes or variations from the paper projection were required, showing remarkable accuracy in the contour work, which is especially noteworthy, when it is recalled that the area was densely wooded and covered with a thick growth of underbrush and laurel. The surveyed area comprised about 2 sq. miles, and the cost of the survey approximated $600, or $300 per square mile.

The other survey was that of a portion of the lands of the Mathieson Alkali Works, at Saltville, Va. This land formerly belonged to the Holston Salt and Plaster Company, a corporation engaged in the manufacture of salt. The Mathieson Alkali Works secured the property for the purpose of erecting thereon a plant for the manufacture of soda-ash and allied products. The contemplated works were to be very extensive and to employ a large force of men. As the old company had no maps of their property, on which the various salt furnaces, salt wells, houses and other improvements were located, the new company decided to have a complete topographical survey made, and this duty devolved upon the writer.

The survey generally comprised the valley of the Mill Spring Branch, as the stream was called locally. In this valley were situated the salt wells, and along the edges the various improvements, the whole valley being rimmed in by hills nearly 300 ft. high. The Saltville Branch of the Norfolk and Western Railroad skirted the west side of the valley. The survey also included part of the valley of the Holston River, on the banks of which the works of the company are located.

Mr. Low. In this case, also, it was desired to have an accurate survey of the property, and especially one which would show the true location of the contour lines. It was an ideal piece of country for a stadia survey, being entirely devoid of timber, the land not occupied being used for grazing cattle. Owing to the many artificial features involved, it was not considered advisable to use the stadia method, but to use instead that generally adopted on railroad surveys.

As at Prince's Flats (or Norton), base lines, marked by stakes set every 100 ft., were run in suitable and selected places. For instance, one was run up the middle of the valley, another along the railroad track, and others were run along the different highways, along the foot of the hills which skirted the valley, some higher up the hill sides, and some along the tops. Care was taken to run lines wherever possible along the ridges or high ground and as nearly as possible at right angles to the slopes. Spur lines were run where necessary along small streams and up ravines. All lines were run with a transit, and, in all but a few instances, formed closed circuits, which usually closed exactly or with a variation of a few minutes only. Each line was marked with a distinctive letter, for the purpose of identification on the ground.

Levels were run over these lines, the elevations of all stations being taken. The levels checked closely, the variations not exceeding a few hundredths of a foot.

The topography was all taken by the offset method, that is, on lines at right angles to the bases, and at the regular stations, placed, as stated, at 100-ft. intervals, and in a few instances at 50-ft. intervals.

All buildings of every kind were located, two measurements to each being taken, one the distance from the base line on range of one of the sides of the building to the corner, and the other from the same corner to the nearest station, the station and plus, where the range cut the center line, being, of course, also noted. The size, character, etc., of the building was also noted. The contours were taken, as at Prince's Flats, by a hand level.

The maps were plotted to a scale of 200 ft. to 1 in., or $\frac{1}{2400}$. The map sheets were 19 x 24 ins. in size, fitted together at the adjoining edges by crosses. The lines were plotted by means of a large paper protractor, using a standard meridian, to which the lines were referred. The main base line, extending from the Holston River to Buena Vista, a distance of about 3 miles, was checked carefully by means of latitudes and departures. As nearly all the other lines were on closed circuits, tied to the main base, the plotting could be checked in those instances, and it usually closed remarkably well. Upon the completion of the maps, a tracing of the whole area was made.

The area surveyed was about 3 sq. miles. The cost was about $900, or $300 per square mile. This is hardly a fair way to express the cost, as

the areas were comparatively small, and, also, on account of the large amount of detail taken, which is not the case in making surveys covering large areas, devoid of many artificial features, where only salient points are taken and the intervening portions sketched in or interpolated. Mr. Low.

JOHN F. WALLACE, President, Am. Soc. C. E.—The speaker was, for a short time, Vice-President and Manager of the Mathieson Alkali Works. One of the first pieces of information which he desired was a map showing the location and physical features of the property, and the results of Mr. Low's topographic survey were brought to him. Mr. Wallace.

The company owned from 13 000 to 14 000 acres of land, and had established, on the banks of the Holston River, a plant for the manufacture of soda-ash and its allied products. The total amount invested in plant and machinery was about $3 000 000.

The works were laid out originally by an English engineer, who had carried out the idea of the English industrial village. Each class of employees had its own settlement. The superintendents of departments had one section of a valley to themselves; the carpenters and the foremen of certain departments had another valley; the laborers in certain parts of the process still another valley; and the ordinary laborers another. The location of the buildings, the side-tracks, the drainage and other improvements, which were being considered and discussed continually, made a map of this kind an essential feature of the economical management of the property.

For the area covered, the speaker has never known of a more complete or accurate topographic survey than that by Mr. Low. He has had occasion to test it in innumerable ways, and, to his pleasure, has never found any errors. In the consideration of various projects for the development of the property, the location of roads, side-tracks, trestles, and apparatus to take care of the waste products, etc., he was able to make profiles in any desired direction, and the assistance given by the information embodied in this survey was so great that the speaker has always felt indebted to Mr. Low for it, and takes this opportunity to pay him a tribute. A great many engineers, like Mr. Low, spend their lives in obscure sections of the country, and we never hear of them. The faithfulness with which they carry out their work is never known. It is with pleasure, therefore, that the speaker calls attention to the accuracy of this particular survey.

HENRY B. MAGOR, Jun. Am. Soc. C. E.—The speaker was engaged in making the survey for the Hankow and Canton Railway, in China, of which William Barclay Parsons, M. Am. Soc. C. E., was chief engineer. Before sailing for China it was a question as to what kind of a survey, or, in fact, whether any survey whatever, could be made. Mr. Parsons, however, decided that a stadia survey was the only one Mr. Magor.

Mr. Magor. practicable. The party consisted of a chief of party, a topographer, an instrument man and two rodmen.

Before leaving Hankow it was considered absolutely necessary to check all the instruments and measure the rods. The latter was done by laying out a long base line on the Bund (the principal street in Hankow). With the assistance of some of the native policemen, and with three days' work, the subdivision of the rods was made exactly right. This was an important preliminary as the readings on the survey were not to be checked, and if the rods had not been correct there would have been a large cumulative error in the total length of the survey, 742 miles.

Stadia readings were taken for the distances, and the levels were taken by stadia readings and vertical angles, and the courses from the compass. The distance was covered in 75 working days, making an average of about 10 miles a day.

At the beginning of the survey a sight was taken on a pagoda, the latitude and longitude of which had been accurately determined to seconds by the Customs Department. At the end of the survey a sight was taken on another pagoda, the location of which had also been similarly determined. The latitudes and departures were then worked out and the total distance checked within less than 2 000 ft., or less than $\frac{1}{2}$ mile in 742 miles, which shows how accurate a stadia survey can be made.

The sights averaged about 1 500 ft., but one (when it was necessary to get away from a crowd in a hurry) was as great as 5 500 ft. Whenever it was possible, solar observations were taken to determine the declination of the needle. The first reading was 45 minutes west, and zero, or the magnetic meridian, was passed when about three-quarters of the distance had been traversed. Thus, fortunately, there were practically no corrections to be made in reckoning the latitudes and departures.

The levels, of course, could not be expected to check, in fact, no attempt was made to check the readings for elevation. The line was merely run from one point to another, the object in taking any levels whatever was to determine any particular gradient over a summit, and this, of course, was accomplished satisfactorily. The levels, however, checked reasonably well with the barometric readings on the summit, which was about 1 000 ft. above the sea; but neither of these results could be relied upon, as they were only approximate.

The stadia rods were 15 ft. in length, and in measuring very long distances half the wire interval was used and then the remainder was determined by the gradienter. Corrections of the distance for differences in elevation were not made. At certain places where the vertical angle was great, the correction of the distance was computed, but was so small that it would have been of no importance on a survey of this kind.

R. S. Buck, M. Am. Soc. C. E.—In the Government survey of the Red River it was endeavored to limit the length of the sights to about 600 or 700 m. The longest sight was about 1 000 m., but in such long sights there was a likelihood of considerable error. Mr. Buck.

All the speaker's stadia work has been done in level country, and he would like to inquire as to what extent corrections for distance are applied in making surveys in very hilly country.

Henry Goldmark, M. Am. Soc. C. E.—In the work in the Rocky Mountains, with which the speaker was connected, the stadia was only used to a small extent. John Q. Barlow, M. Am. Soc. C. E., did the instrumental work and made some very rapid surveys by that method He also surveyed some of the auxiliary reservoirs (not yet built) of the Ogden power plant by stadia, but the principal reservoir site was surveyed by the ordinary railroad methods, that is, by base lines and by secondary lines at right angles to these bases, all elevations being taken with a level. Mr. Goldmark.

There is no doubt that the stadia method would have been much more rapid and economical than the method adopted for that survey, and would have been sufficiently accurate. The surveys were very good, but the speaker has always considered them as more expensive than was necessary.

A. J. Himes, M. Am. Soc. C. E.—In making the United States Deep Waterways Surveys for a 30-ft. canal along the Oswego-Mohawk route, the territory was divided into two parts, known as the Eastern and Western Divisions, and the survey of each part was made by a separate corps of engineers. The dividing line was at Herkimer, a few miles west of Little Falls. Mr. Himes.

The route, on the Western Division, followed the Oswego River, Oneida Lake, Mohawk River and connecting lines, a total distance of about 91 miles. The work on this division included the necessary harbor surveys at Oswego, soundings in Oneida Lake and the various streams, the triangulation of Oneida Lake, covering an area of about 78 sq. miles, a complete system of test borings and about 120.7 sq. miles of topography. The topography was taken with the stadia, in accordance with instructions issued by the United States Board of Engineers on Deep Waterways.

A base line was measured along the route, and Oneida Lake was covered with a chain of quadrilaterals. This base line and triangulation formed the foundation of the survey.

Method.—The manner of taking the topography was: To start from an instrument point on the base line, with the base-line azimuth, and run a circuit through the territory to be covered, returning, generally within a distance of 2 miles, to the base line and checking the azimuth on the nearest base line point.

The courses in the circuit were all measured with the stadia, except-

Mr. Himes. ing those belonging to the base line, which necessarily formed a part of the closed circuit.

The distances and vertical angles were read twice, once as a foresight and once as a backsight, and the mean of both was used in plotting.

As each instrument point was occupied in running the circuit, readings were taken all around the circle, on all objects of topography, natural or artificial, which could have any bearing upon the proposed plans and estimates.

A sufficient number of readings was taken to permit the development of contours having a vertical interval of 2 ft.

All buildings, roads, railroads, property lines, ditches, canals, streams, high-water marks, orchards, swamps and rock outcrops were located, and notes were made of the character of the soil and substrata where exposed.

Sketches were carefully made in the field books, and were found to be of great service in plotting. Each stadia reading was numbered, beginning each morning with number one, and the locations of the principal readings, with their numbers, were shown on the sketches.

Each party was supplied in advance with a list of data for use in the locality where it was at work, which included the number, elevation, latitude and departure of all base-line instrument points, the true azimuth of the base-line courses and the approximate boundary of the territory to be surveyed.

With this information, it was an easy matter to find a starting or a closing point for a circuit, and to check the circuit after it was run.

In order to avoid complications in the adjustment of circuits for plotting, they were made as simple as possible. Along the base line, their length was limited to 2 miles. A secondary circuit was allowed to start from and return to a primary circuit, but no circuit was allowed to start from one circuit and close on another, or start from a circuit and close on a base line, save in exceptional cases.

Checking Survey.—In the beginning of the survey, the field work was checked by having each party plot its own work roughly on protractor sheets. At that time there were only two stadia parties at work. They were located very near together, and the checking was fairly well done. It was found, however, that to do the work thoroughly would require the addition of a draftsman and a computer to each party. Even with this change it would have been difficult to keep the checking close up to date, because the book which was in the field during the day was always needed at night for reducing the readings and completing the notes, so that the draftsman either had to be a whole book behind in his work, or else the field party must work first in one book and then in another, either of which methods was very objectionable.

Working Maps.—After the survey had been in progress for a time, it was decided by the Board that since working maps were needed,

they could as well be made by the field parties instead of the plotting of the protractor sheets, thus saving the labor on one set of maps and securing the necessary check on the survey and a complete set of working maps at the same time. Mr. Himes.

It was desirable to plot the base line on these working maps by latitudes and departures, and, as they must be figured, it became convenient to check the stadia circuits by figuring their latitudes and departures and comparing results with those of the base line at the closing station.

The method pursued thereafter was, therefore, to check the stadia circuits by latitudes and departures immediately when closed, to send a book, when completed, to the headquarters of the survey, where the numerical work contained therein was first checked and the notes then plotted and errors or omissions reported back to the field for correction. The work was so divided among the stadia parties that each party could remain long enough in one locality to fill several books, and it was very seldom that a party had to go back to a previous location to look up errors.

After the winter work on Oneida Lake was completed, the force was so large, numbering for several months nearly 100 men, and the work was so scattered, that it would have been impossible to maintain a thorough control of it if the mapping had not been centralized and reduced to a system as above described.

Each night, after the men had returned to their boarding places and eaten their supper, the notes taken during the day were reviewed, the stadia measurements, when necessary, were reduced to the horizontal and the differences of elevation computed. The latitudes and departures of the circuit courses were figured, and if a circuit had been completed, the latitude and departure of the closing station were compared with its latitude and departure as given in the base-line notes. By this means, when errors occurred, they were always discovered promptly and corrected before they could create any confusion in the office work.

TABLE No. 2.

Error.	No. of circuits.	Maximum length of circuit.	Minimum length of circuit.	Average length of circuit.
Exceeding $\frac{1}{100}$	2	4 240	3 162	3 701
Between $\frac{1}{300}$ and $\frac{1}{700}$	4	26 842	1 486	13 076
Between $\frac{1}{700}$ and $\frac{1}{1000}$	10	31 009	2 140	12 737
Between $\frac{1}{1000}$ and $\frac{1}{1500}$	7	7 015	1 813	4 779
Between $\frac{1}{1500}$ and $\frac{1}{2000}$	6	56 048	2 589	21 163
Between $\frac{1}{2000}$ and $\frac{1}{3000}$	7	31 627	4 597	11 555
$\frac{1}{3305}$	1	.....	.....	16 522
$\frac{1}{5514}$	1	.....	.....	5 324
$\frac{1}{10965}$	1	.....	.....	4 385
$\frac{1}{17104}$	1	.....	.....	13 683
Average error $\frac{1}{1066}$	40	56 048	1 486	11 732

Mr. Himes.

Precision of Work.—In order to show the degree of precision which was attained with the stadia, all circuits throughout the whole work which were closed on their starting points have been reviewed and their closing errors computed. The results are shown in Table No. 2, from which it may be seen that the error can readily be kept below $\frac{1}{700}$.

The error in elevation seemed to bear no relation to the length of the circuit. It seldom exceeded 1 ft., and was generally less than 0.5 ft.

Organization of Party.—A stadia party usually consisted of five men; an instrument-man, a recorder and three rodmen. Sometimes a laborer was added to a party, temporarily, for the purpose of cutting brush or rowing a boat.

The instrument-man was required to direct the work of the party and attend to its needs. He selected the boarding place, paid incidental expenses and rendered accounts for board, team hire and sundries to the Assistant Engineer. He also made weekly statements of the amount of work done, and kept a journal in which were recorded all events which might have a bearing on the progress of the work, such as storms, high water, the transfer of men from one party to another, and the loss of tools.

In the field he either kept notes or ran the instrument and had the general direction of the operations of the party.

The recorder acted as an assistant to the instrument-man, recording notes, running the instrument or taking charge of the party, temporarily, when the instrument-man desired to make a reconnoissance, or for some other reason was separated from the party.

The work of the rodmen was to carry the stadia boards in the field, to select points where readings were to be taken, and report to the instrument-man any information they might obtain which would be of value to the survey.

An effort was made to allow each man in the party to acquire as much experience as possible, and for that purpose the recorder and instrument-man exchanged work occasionally with the rodmen. While the progress of the party was hindered a little at such times, it is believed that by increasing the interest of the men in their work the final result was benefited.

Field Notes.—Each instrument-man was required to keep his notes so clearly that any stranger, not familiar with the work, could read them readily. For this purpose, each book contained an index and a brief synopsis of its contents, and all notes copied from other books were thoroughly cross-referenced. For instance, when a stadia circuit was started from a base-line instrument point, a reference was made in the notes to the book and page of the base-line notes where the station was recorded, and also to the place in the level notes where the elevation was found.

No erasures were allowed in the notes, and the date, the number of the instrument, and the names of the observer and note-keeper were always recorded. Mr. Himes.

After the computations had been made, references were made to the computation books where the figures were recorded.

It was found nearly impossible to keep the notes written up and the reductions made by evening work, so every stormy day was used to catch up, and, on rare occasions, the whole party remained inside for a day to work on the notes.

Practice developed many convenient details in doing the work, which aided much in the interpretation and plotting of the notes. Some of the features of the work which have been stated were not begun until the work had been in progress two or three months, as they were the outgrowth of experience.

Instruments.—The instruments used were the usual railroad transits with stadia wires. There were two made by Buff and Berger, two by Seelig, and one by Heller and Brightly. In the latter instrument the wires were adjustable, in the others they were fixed, but in all cases the value of the interval was determined, and the rods graduated accordingly, no attempt being made to adjust the wires in the Heller and Brightly transit.

Reduction of Readings.—The rods were graduated and the reductions made according to the formula deduced by S. W. Robinson, M. Am. Soc. C. E., of Michigan University. The tables computed according to that formula by Alfred Noble, M. Am. Soc. C. E., and Mr. Wm. T. Casgrain were used at first in making the reductions. Later, charts for doing the work were furnished by the Board, and their use greatly lessened the amount of necessary labor.

Graduation of Rods.—The style of graduation used on the rods was found to be especially adapted to the work, and is, therefore, exhibited in Fig. 4. The features of especial merit are, its simplicity and absence of fine detail, which avoids confusion, and the use of sharply-defined angles for divisions, which permits very precise readings.

Fig. 4.

The rods were 4 ins. wide and 12 ft. long. The 5 and 10-ft. marks were painted red. Sights 2 000 ft. long were read very frequently on instrument points as well as for side readings.

Seasons.—The work was begun October 23d, 1897, and prosecuted continuously by one or more parties until its completion, November 5th, 1898. The work has been done in all seasons, in all kinds of weather, and in many kinds of country, including the wharves, buildings, bridges and canals in Oswego, the railroad yard in Utica, the irregular and thickly

Mr. Himes. wooded hills of Sand Ridge and the open, level plains of the Mohawk Valley.

As to the comparative merits of the seasons for taking the topography, it was found that July and August were the most unfavorable months, and that the best time is in the early spring or late fall, when the weather is cool, the trees are bare and there are no crops or heat waves to interfere with the sight or impede the progress. In warm weather the atmosphere is too unsteady to permit long sights, the foliage is thick, and obstructs the view, and the summer heat renders the traveling more fatiguing, so that the rodmen cannot cover as much ground.

The progress of the party is frequently dependent upon the speed with which the notes are recorded, and in extreme cold weather the progress is lessened somewhat by the difficulty of using a pencil with cold fingers. In other ways, no especial difficulty was experienced in the winter time. During the winter of 1897–98, the snow caused no more interference with the work than did the rain during the following summer.

The average number of days lost each month by the several parties, on account of storms, was as follows:

1897, October, 0.0; November, 3.5; December, 4.5.

1898, January, 1.0; February, 4.4; March, 1.4; April, 2.8; May, 4.7; June, 1.2; July, 0.7; August, 3.2; September, 1.5; October, 3.6.

Cost.—The cost of the stadia work per square mile, using the same items given by Mr. Landreth, viz.: salaries, maintenance, traveling expenses and supplies, was as follows:

Field work	$179
Mapping	101
Total	$280

There have been excluded from the area several swamps which were surrounded with stadia work, but not penetrated. Their aggregate area is 13.4 sq. miles.

The average number of readings per square mile was about 1 440, or 2¼ per acre.

The minimum average area covered per day by one party on a single piece of work was 0.058 sq. mile. The maximum was 0.257 sq. mile. The average area covered per day by one party for the whole survey was 0.123 sq. mile.

In computing these averages, holidays and other lost time has not been deducted from the total number of days in the field.

About 83% of the area was mapped on a scale of 1:5000. The remainder was mapped on a scale of 1:2500.

Mr. Himes.

The cost of mapping includes plotting soundings, test borings and triangulations. There were upwards of 12 sq. miles of soundings, and the cost of plotting these cannot be separated from the rest of the mapping. It would, perhaps, be more accurate to call the total area of mapping 133 sq. miles. The cost per square mile would then be $92. A vertical interval of 2 ft. was used in plotting the contours.

All elevations, determined either by sounding or with the stadia, were inked on the maps. This, it is thought, was not done in Mr. Landreth's work.

Conventions furnished by the Board were followed throughout the work, but while they were used sufficiently to show clearly the topography of the country, care was exercised to avoid everything in that line not wholly necessary, to the end that the maps might be a useful medium for an engineering study rather than an exhibition of artistic skill.

In making use of the foregoing data, it would be well to note that the maximum rate of progress was about four times the minimum. Also, in Mr. Landreth's data, the field work on Fish Creek for 5-ft. contours cost less than that for 10-ft. contours on Salmon River, while, on the Black River, the cost for 10-ft. contours was one-fourth of that on Salmon River.

It is surprising that on Fish Creek, where 620 readings per square mile were taken, the cost should be nine-elevenths of that on Salmon River, where 256 readings per square mile were taken.

On the Black River, Mr. Landreth took 131 readings per square mile, which is about one reading for every 5 acres. Eighty-five sq. miles were covered in that way. This small number of readings and the low rate of cost make it interesting to consider the cost of the United States Geological Maps, and to inquire whether work done in that manner would have served the purpose. In the New York State Engineer's Report for 1895, it is stated that the total cost of the United States Geological Maps in that State, prior to 1895, is at the rate of $10.62 per square mile.

It is noted in Table No. 1, that a large percentage of the work was taken directly from base-line stations. That practice was forbidden on the main surveys, it being considered that greater chances for error were afforded than where the topography was all taken from stadia circuits.

Considering the rapid movement of Mr. Landreth's party, it was undoubtedly better for him to use a large party and do the plotting at once.

The method of plotting outlined above was the outgrowth of conditions which had to be met, and should never be adopted without due consideration.

The results of the work appear to warrant a more general use of the stadia on preliminary surveys.

Mr. MacGregor. R. A. MacGregor, Assoc. M. Am. Soc. C. E.—In the survey of Baltimore, Md., the stadia method was used on more than 33 square miles. The maps were plotted to a scale of 200 ft. to 1 in., and all fences, roads, houses (with some details of the houses), contours at 5-ft. intervals, wooded areas, creeks, pasture and cultivated areas, etc., were shown.

The method used was somewhat similar to that described by Mr. Himes. The instruments were provided with fixed stadia wires; the horizontal and vertical circles were divided to read to 30 seconds, and the vernier of the vertical circle carried a level tube, each division of which had a value of 30 seconds.

The stadia boards were T-shaped in section and the figures were similar to those used on the United States Lake Survey. The meter was used as the unit of length on account of the convenience in using the tables for difference of elevation, which were those computed by Messrs. Ockerson and Teeple of the United States Lake Survey, and which give the elevation in feet for the distance in meters.

Everything was plotted in the field, the map being made complete in pencil before leaving the locality. The shots were taken and reduced by the recorder, and plotted by the draftsman, who stood behind him. The topographer in charge then did the sketching.

Each sheet contained $\frac{1}{4}$ sq. mile of territory, and had at least five, and sometimes a great many more, points plotted thereon, before being sent into the field. The co-ordinates of points and azimuths and distances between the points were given, and closed circuits were run between them. The circuits were computed in the field and no error greater than 1 m. was allowed, this being about the limit of accuracy in plotting to 200 ft. to 1 in.

Elevations were read and plotted to 0.1 ft., and the level runs usually checked within 0.1 ft. On a run of 4 370 m. the error was only 0.18 ft. between points determined by spirit leveling.

The average error of closure was about 1 in 700 between points determined by precise traverse methods in which the error was 1 in 10 000. On a traverse of 1 000 to 1 200 m. the error was occasionally 1 in 1 200. These ratios are all by actual calculation.

The cost of the 33.3 sq. miles was about $850 per square mile for the stadia field work alone. The average number of shots to the acre was about 10. The work done by the speaker, about 3 500 acres, averaged 18 acres per day. The maximum number of acres surveyed in a day was about 75, with 207 shots, the minimum a small fraction of an acre, as the character of the territory varied considerably. There were small clusters of houses on most of the sheets, but on one sheet in the most thickly settled portion, there were 840 houses and 4 000 shots. The number of shots per day averaged 180, with a maximum of 349, all being plotted and the sketching done.

General H. T. Douglas, M. Am. Soc. C. E., was Chief Engineer of the survey, and Mr. O. W. Connet was in charge of the topographical division. Mr. MacGregor.

Kenneth Allen, M. Am. Soc. C. E. (by letter).—The cost of topographical surveys is of little significance without full knowledge of the character of the ground, the extent of the survey, and such necessary expenses as transportation and board, which depend on the locality. Mr. Allen.

Of the remaining factors, the scale of the map, which generally controls the limit of error, is of prime importance. So that, when it is claimed that a certain survey was well conducted because costing less per unit of area than another, it means absolutely nothing. For this reason, such details as are given by the author are of value in enabling one to estimate the cost of similar surveys under similar conditions.

In connection with the recent investigations of the Baltimore Sewerage Commission, various topographical surveys were conducted by the writer, from which the following five are selected, data regarding the cost of which are given in Table No. 3, and are, perhaps, of sufficient value to place on record.

The ground being similar in Surveys I and II, a comparison of their costs is of interest in showing the economy of reading compass bearings and setting up on alternate points, where this is sufficient, and practicable.

A comparison of the cost of Survey I with the costs of Surveys IV and V shows the increased expense due to plotting on a larger scale and halving the contour interval, the territories being fairly open in each case, and I and V having but gentle slopes. It will be noted, too, that IV and V are quite limited in extent in comparison with the others, which in part accounts for their greater cost per square mile.

TABLE No. 3.

Survey	I.	II.	III.	IV.	V.
Contour interval, feet	5	5	5	2½	2½
Scale of map	800′ = 1″	800′ = 1″	400′ = 1″	200′ = 1″	200′ = 1″
Actual working days	13	21	61	16	14
Total area, square miles	2.04	2.75	4.83	0.823	0.733
Area per day, square miles	0.157	0.131	0.079	0.0515	0.0524
Salaries per square mile	$54.90	$78.00	$140.20	$323.61	$256.21
Expenses per square mile	11.91	16.49	28.54	30.73	13.50
Cost per square mile	$66.81	$94.49	$168.74	$354.34	$269.71
Estimated cost per square mile, including superintendence	.....	.....	186.57	363.18	284.01

All these surveys were made by stadia and vertical arc measurements, running in circuits between established points, and referred to benches established by the wye level. The bearings were determined

Mr. Allen. by the compass, except in Survey II, where the azimuths were turned off by vernier. The times given are for days worked only, and do not include time lost; while the costs given are for work in the field only, and do not include the map work done in the office. They do include, however, all expense for field work, including the plotting of the map on field sheets which were turned in to the main office and transferred by the draftsman to a sheet on which the lines or points of control had been plotted.

Surveys III, IV and V were made with considerable care and detail, being for the purpose of laying out filtration beds or plants for the treatment of the city's sewage.

Of Survey III, 47% was in timber, and 3% in marsh and water.

Of Survey IV, 12% was in water surface.

Of Survey V, 27% was in timber, 12% was in open marsh, and 6% was in water surface.

In Survey III the range of elevation was 125 ft. Some of the ground was quite rough and filled with a dense thicket of briar and underbrush. In the other areas the range of elevation did not, generally, exceed 40 ft.

The greatest error in closing the circuits of Survey III on the map, averaging about 1½ miles in length, was 80 ft., occurring in an unimportant place; the next was 30 ft., and the average 10 or 15 ft.

Two steel tape lines, which had been run through this area, were utilized as bases; while in Survey V, steel tape base lines were run from known positions determined by the United States Coast and Geodetic Survey.

The writer is interested in noting the use of a megaphone in the field. The means of communication between the rodman and the man at the instrument should be effective at the maximum distance and not liable to misinterpretation. A simple code of signals is believed to be preferable to verbal orders for directing the ordinary movements of the rodman, and for this purpose a whistle or small flag may serve; but there are times when some means of conveying fuller directions than can be done in this way is very desirable, and the megaphone may answer the purpose. Sight signals have the advantage of being effective where there are disturbing noises from winds, factories or waterfalls, while signals by sound may be preferable where there is more or less obstruction to the sight.

Mr. Fisher. Wager Fisher, Jun. Am. Soc. C. E. (by letter).—The cost of the field work for topography with 10-ft. contour intervals on two territories, which were, respectively, 65% and 25% wooded, is given by Mr. Landreth as $66 and $16.50 per square mile. These results present an opportunity of determining, in a measure that is at least suggestive, how much the cost of such topography varies with the percentage of woodland.

Mr. Fisher.

The writer, having computed the cost of a similar survey made in the summer and fall of 1899 by the New York State College of Forestry* of their tract in the Adirondacks, in Franklin County, N. Y., submits the results in order that with others they may form a basis of comparison.

The territory was practically all wooded, and presented the usual difficulties of the mountainous and hilly country of the Eastern States. The area covered was 32 sq. miles, and, in the effort to find the cheapest method, both rectangular and irregular systems of stadia transit lines were tried. There were 172 miles of stadia circuit. The survey was made between July 1st and December 1st, 1899, but not continuously. Comparing the seasons, summer and fall, differences in foliage were largely counterbalanced by differences in weather. The cost of maintenance was about 41% of the total expense.

TABLE No. 4.

Survey.	Percentage of woodland.	Cost per square mile.
Black River	25	$16.50
Salmon River	65	66.00
New York State College of Forestry	100	85.00

Mr. Christian.

G. L. Christian, Assoc. M. Am. Soc. C. E. (by letter).—The writer fully agrees with Mr. Allen that the cost of topographical surveys depends upon the character of the ground, the extent of the survey, the scale of the map, transportation facilities, board, etc. The season of the year is also a very important factor.

For instance, in the Middle Atlantic States, the work can generally be performed to much greater advantage early in the spring or late in the fall, especially if the district is heavily wooded.

In 1896 the writer made a survey of a 57-acre tract of land in Westchester County, N. Y., which the owners wished to subdivide. Most of the work was done during the month of July, and the weather was exceedingly hot and sultry.

Three-quarters of the tract was woodland, with much thick underbrush and heavy foliage. The land was much broken, and varied from a rise of 2 ft. in 100 to a rise of 40 ft. in 100, and had a total rise of 150 ft.

The transit lines (12 750 ft.) were first run so as to cover what seemed to be controlling points, such as along the tops of ridges, at the foot of hills, etc., stakes being set every 50 ft.; after which levels were carefully run over the whole line with a wye level, elevations being taken at each stake, and at intermediate points if necessary.

* Doctor B. E. Fernow, Director. Field work supervised by Professor H. N. Ogden, Assoc. M. Am. Soc. C. E., and Mr. William E. Mott, both of Cornell University.

Mr. Christian. From these lines and levels the 5-ft. contours were located on the ground, at right angles to the transit lines, with a Gurley hand level, an ordinary level rod and a steel tape.

The actual working time of the field party each day was from 9 A. M. to 5 P. M., with one hour for lunch. The work was 2 miles from the office, and transportation, consisting of a horse and wagon, was furnished each way.

The net cost of the field work, transportation, stakes, and mapping was $1.47 per acre, divided as follows:

Field work including transportation:

Running transit lines	$0.40	
" wye levels	0.19	
Five-ft. contours, with hand level	0.53	
Total		$1.12
Stakes		0.07

Mapping:

Plotting transit lines	$0.13	
" contour "	0.15	
Total mapping,		0.28
Grand total		$1.47

The map was plotted on a scale of 100 ft. to 1 in.

Mr. Rafter. GEORGE W. RAFTER, M. Am. Soc. C. E. (by letter).—As to the cost of the several surveys it may be remarked that, without a full statement, Mr. Landreth's tabulation is somewhat misleading. Thus the total cost of Black River survey was $3 815.47; Salmon River Reservoir, $2 459.05. The cost of Fish Creek Feeder is included in the general cost of feeder lines, which is $13 013.23. It should be understood that the cost of Fish Creek Feeder is for topographic work only; all expense for leveling of every kind is included elsewhere. An understanding of these facts gives to the author's results a somewhat different complexion than they would have if unexplained. The writer, however, wishes to accord Mr. Landreth very high praise. The Salmon River and Fish Creek surveys were largely made during severe winter weather, and are evidence of his great energy, as well as skill in this special line of survey.

Another matter which contributed to the success of the work may be mentioned. Each chief of party was required to report to the office at the end of each week exactly what had been done. These reports took the form shown in Table No. 5, which illustrates Mr. Landreth's work on the Salmon River Reservoir for the month of November. At the end of the month the weekly reports were compiled into a monthly report, and forwarded to the Board.

TABLE No. 5. Mr. Rafter.

REDFIELD, N. Y., December 1st, 1898.

Report of progress on Survey for Salmon River Storage Reservoir for month of November, 1898.

By WILLIAM B. LANDRETH.

Date.	Stadia.		Distance.		No. Days actual work.	Weather.	Character of ground.
	Stations.	Shots.	Stadia Line.	Base Line.			
Nov. 1	20	110	5 326	3 000	1		
" 2	26	147	6 942	5 400	1		
" 3	15	156	3 705	7 600	1		
" 4	18	146	5 234	3 200	1		
" 5	28	90	11 581	5 600	1	Rain.	
" 6	....	....	.....	.....	....		
" 7	22	51	5 285	4 300	1		
" 8	21	74	4 763	2 000	1		
" 9	22	38	4 719	2 090	1	Rain.	
" 10	16	20	3 072	.....	1	Rain.	
" 11	20	19	4 535	.....	1		
" 12	22	28	5 652	.....	1	Rain.	
" 13	10	9	3 754	.....	1		
" 14	22	27	11 834	3 500	1	Rain.	
" 15	22	44	7 845	.....	1	Rain.	
" 16	17	13	5 787	3 000	1		
" 17	25	20	13 275	.....	1		
" 18	20	57	13 501	3 000	1		
" 19	15	92	24 788	.....	1	Rain.	
" 20	21	37	9 075	.....	1		
" 21	17	65	6 225	14 800	1		
" 22	14	60	8 139	.....	1		
" 23	21	173	11 043	10 100	1	Snow.	
" 24	20	101	6 587	9 600	1		
" 25	....	....	.....	.....	....		
" 26	21	109	8 700	5 700	1	Snow.	
" 27	18	143	14 890	4 200	1		
" 28	17	158	3 838	14 400	1		
" 29	19	141	6 657	10 300	1		
" 30	23	63	8 420	14 700	1		
Totals....	552	2191	225 262 42.66 Miles.	126 490 23.95 Miles.	28	.....	Estimated: 35% Open, 65% Wooded.

W. B. LANDRETH, M. Am. Soc. C. E. (by letter).—Referring to Mr. Buck's inquiry regarding corrections for distance in stadia surveys in hilly country; the practice on the work described was to make corrections for distance on all readings between stadia points used in calculating circuits and to disregard them on all shots where the correction was less than 3 ft. Mr. Landreth.

The stadia reduction diagrams used were so drawn that the approximate reduction could be noted at a glance and taken off closely if necessary with little labor. All elevations determined were noted on the maps, as in the general work of the U. S. D. W. Survey, and Mr. Himes has been misled regarding them by their having been omitted by the engraver from the illustrations accompanying the paper, owing to the very small scale used.

Mr. Landreth. The writer is unable to give an explanation for the variation in the cost of the three surveys that is entirely satisfactory to himself, but believes that the percentage of wooded area has a great influence on the cost of the surveys. On the Salmon River and Black River Surveys, with a 10-ft. contour interval, the former, with the larger percentage of wood, cost the most, while the Fish Creek Survey with a wooded area about midway between the others, but with a 5-ft. interval, cost 82% of the former.

The methods used on the U. S. Geological Survey could not have been used to advantage on the Salmon River and Fish Creek Surveys owing to the necessity of doing the field work in the late fall and winter, and it is doubtful if there would have been any saving in cost by using that method on the Black River Survey owing to the 10-ft. interval used, as compared with the 20-ft. interval on the Geological Surveys.

To avoid a possible misunderstanding of the items covered by the table of cost of surveys noted by Mr. Rafter, the writer would state that the cost of the following work is included in the amounts given: Salmon River and Fish Creek surveys, stadia field work and map work; Black River, base line and levels on two-thirds of the territory, stadia field work and map work on all the territory.

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TRANSACTIONS.

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No. 881.

THE NINETY-SIXTH STREET POWER STATION OF THE METROPOLITAN STREET RAILWAY COMPANY, OF NEW YORK CITY.

By L. G. MONTONY, Assoc. M. Am. Soc. C. E.

PRESENTED SEPTEMBER 19TH, 1900.

WITH DISCUSSION.

The events which led up to the building of the new power station of the Metropolitan Street Railway Company were brought about largely by the steady growth of the company.

In 1891 it was decided to adopt mechanical traction for Broadway, and the cable was selected as offering the best tractive power for heavy loads. As the company increased in size, cable extensions were built on Lexington and Columbus Avenues. Having built these, and having laid out other proposed roads, it was seen that the cable, by its multiplicity of sheave pits, curve pits and power houses, was too expensive and cumbersome to be adopted for a large system; and, as the overhead trolley, which had proven so satisfactory in other cities, was prohibited on Manhattan Island, the company began a search for some other system which would prove satisfactory in operation and which would also overcome the objectionable features of both the foregoing systems.

After a careful study of all the underground conduit roads in existence at that time it was decided to build and equip several miles of conduit electric road, experimentally; making the road-bed so that it could be easily changed to a cable road if the experiment failed. This experimental line was built in 1895, and proved so satisfactory that it was decided to adopt the conduit electric trolley as a standard for all main lines. The company now operates all the lines in the Boroughs of Manhattan and The Bronx. Each year, since 1897, it has converted several roads into the conduit electric construction, and when the conversion of the present cable roads is completed, nearly all the main lines will be of this type. All the street railroads in The Bronx are of the overhead-trolley type. The cross-town lines down town are as yet unequipped, because the underground trolley does not seem to be suitable, and the company is still conducting experiments with a view to securing a motive power for them.

Location.—The station is located on First Avenue between Ninety-fifth and Ninety-sixth Streets, and the property extends to the East River. The dimensions of the building are shown on the plan, Fig. 1. Provision has been made for a marginal street along the river front, by leaving a space about 75 ft. wide between the building and the river, in case the Dock Department should ever conclude to condemn and purchase that strip.

The engine house is on the north and the boiler house on the south side of the building. The width of the former is about 112 ft., and of the latter about 88 ft. The chimney is near the center of the boiler room. Fig. 2 is a longitudinal section of the boiler house, Fig. 3 a longitudinal section of the engine room, and Fig. 4 a cross-section through the building.

The space opposite the chimney, in the center of the boiler house, is utilized as follows: In the basement this space contains the water supply piping; on the first floor it contains the feed pumps; the second and third floors are left for secondary heaters; the fourth is for an oil room, and the fifth is used as a machine shop for light repairs. An elevator and a stairway provide access to each floor.

On each of the first, second and third floors space is provided for two large blowers to be used for forced draft if it be found advisable or necessary to force the boilers.

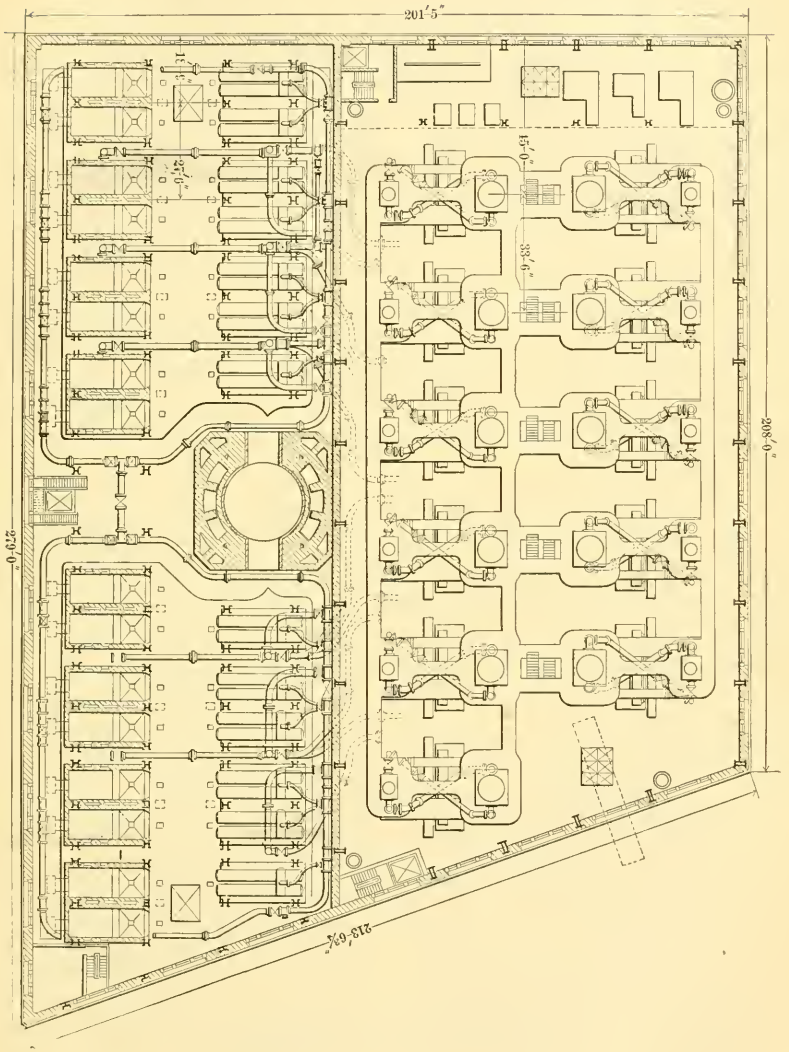


Fig. 1.

At the southeast corner there is a space divided up into a stairway well and a wash room on each boiler house floor for the men. Over each wash room there is a mezzanine floor for lockers, etc.

The Foundations.—Piles were driven over the whole site and then a thick bed of concrete was put in. The thickness of the concrete under the chimney is 20 ft.; under the center of the boiler house it is 7 ft., and under the remainder of the building 5 ft. The piles are from 35 to 40 ft. long, and extend up into the concrete 1 ft. Those under the chimney are 45 ft. long. The piles were not driven entirely to refusal, yet they are firmly embedded in the coarse sand or gravel, and, according to the *Engineering News* formula, they have a sufficient bearing power.

Near the present westerly property line of the site there was an old bulkhead crib, and old cribs extended out along Ninety-fifth and Ninety-sixth Streets, formerly making piers near these streets. The site of the building was formerly the water slip, and therefore the upper part of the excavation was mostly filled material of comparatively recent date.

The cribs extend down near each street line, and are part way under the building. It was found that, by removing the tops of these cribs, when the excavation was being made, it was possible to drive piles somewhat regularly and sufficiently well through the bottom part. Some of the piles were shod. The cribs did not extend under the area occupied by the chimney. The preliminary borings indicated that the space between the cribs was filled with heterogeneous matter down to about Elevation — 15. Under this, and down to about Elevation —75, there were beds of sand, each about 10 ft. in thickness, and growing coarser and more like hardpan as the depth increased. The strata below the cribs seem to be almost level over the whole site. Below Elevation —75 stiff blue clay extends down indefinitely as far as known. The piles were driven on nearly 3-ft. centers under the walls, and on 30-in. centers under the center of the building. Under the chimney they were driven a little closer, being placed so that the triangle formed by each three adjacent piles has 30 ins. on a side. The elevation of the former surface of the ground was from + 6 to + 7 ft. The elevation of the basement is zero.

A vault has been built under the sidewalk on Ninety-fifth Street and on First Avenue, with the floor at Elevation — 2. This will be used for

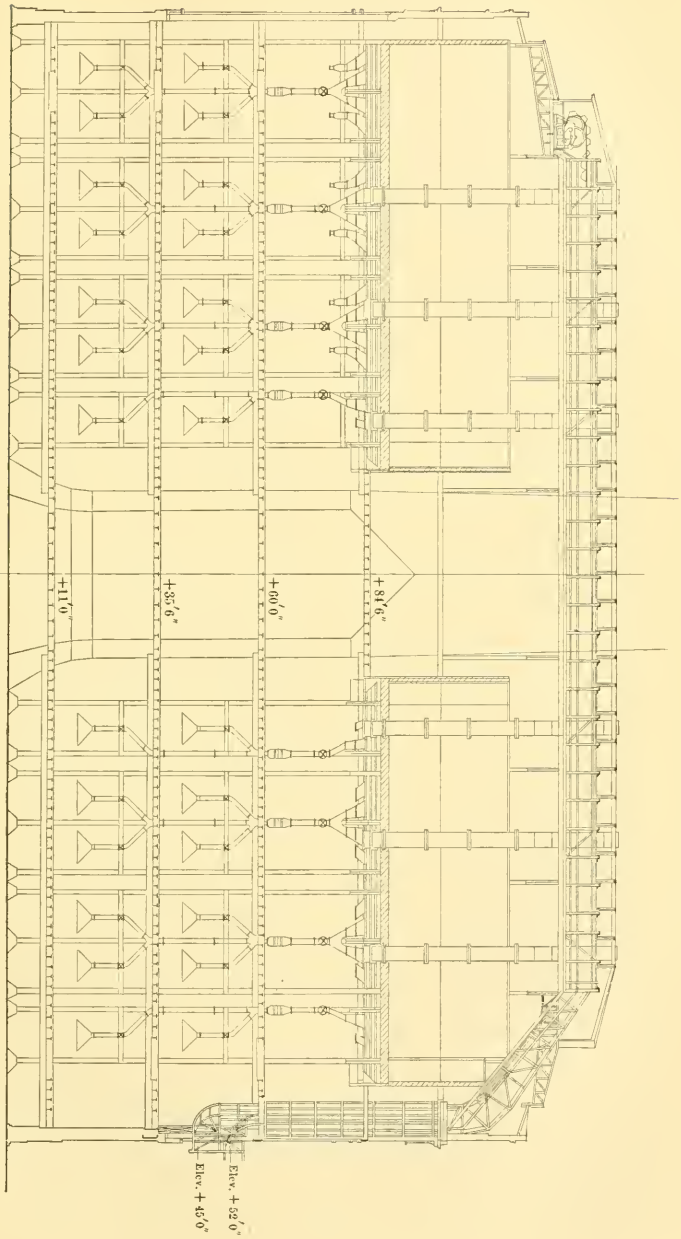


FIG. 2.

the distribution of feeder wires and probably for a small storage battery. There is piling under the wall, and a thick bed of concrete for the floor, which is not piled.

The engine foundations are shown in Fig. 2, Plate XI. They are mostly of brick, but the lower part, below the holding-down bolts, consists of a shell of brick filled in with concrete. Each of the large foundations would contain about 500 000 brick if entirely built of them.

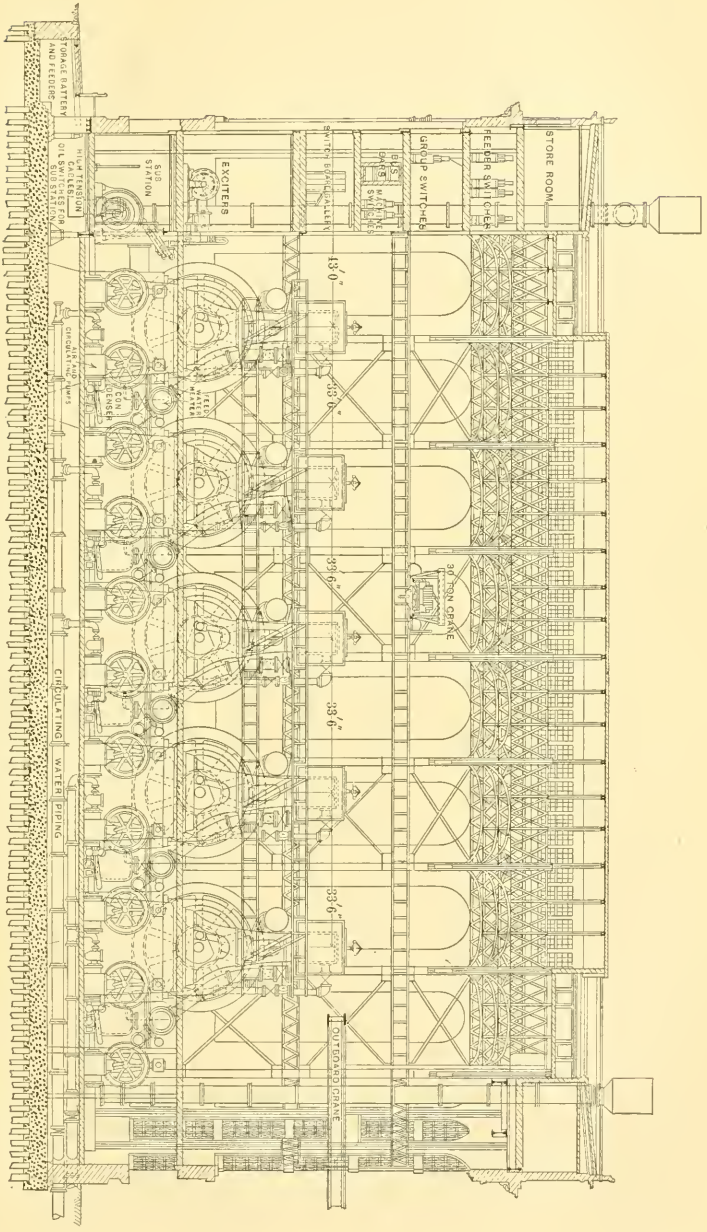
The Engine House.—The engine house has a sub-floor at Elevation + 10, the engine floor being at Elevation + 29. At the west end there is a series of floors at Elevations + 54, + 77 and + 91. These are 22 ft. wide, and are to be used as switch-board galleries. At the top there is a pent house running around on the four sides of the engine house at Elevation + 102.

The engine house has large headroom, and is trussed across by heavy steel trusses 22 ft. 8 ins. apart. These carry a lantern in the center and the pent house on either side.

At the switch-board end, at one corner, there is an elevator and stairway for visitors. At the other corner of the switch-board galleries a spiral stairway is provided especially for the use of the employees. At the other end of the engine house, and in one corner, there is an elevator and stairway in a brick enclosure running up to the pent house and storeroom, with doors to the engine-house floor and each successive floor of the boiler house.

Chimney.—The internal diameter of the chimney is 22 ft., and its height is 353 ft. It is higher than any chimney in this country, and of larger diameter than any in the world. It was built the summer before the building, except the foundation walls. In order to keep at work as many bricklayers as possible, it was built from both inside and outside scaffolds, up to a height of 125 ft., and two material elevators were used. From that elevation to the top a scaffold and elevator was carried on the inside only.

It is built mostly of common brick, and contains a lining of fire-brick extending about 50 ft. above the highest flue opening. It has two shells. The inner is just thick enough to support its own weight, including the fire-brick lining; the outer carries its own weight and the horizontal wind load, taken at 50 lbs. per foot. To resist this overturning moment, and to stiffen the outer shell, counterforts or



LONGITUDINAL SECTION THROUGH ENGINE ROOM.

LOOKING NORTH

Fig. 3.

ribs were built into it, as shown in Fig. 5. These reach within about 2 ins. of the inner shell and form a guide for its movement by expansion or contraction. The capacity of the chimney was figured by Rankine's formulas. Brick was used instead of steel because masonry is the more durable material. The chimney was carried high in order to avoid the use of exhaust fans or blowers, and because it was thought better to let the foul gases escape at as high a level as possible. It contains about 3 400 000 brick, and rests on a block of concrete 20 ft. thick and 88 ft. square.

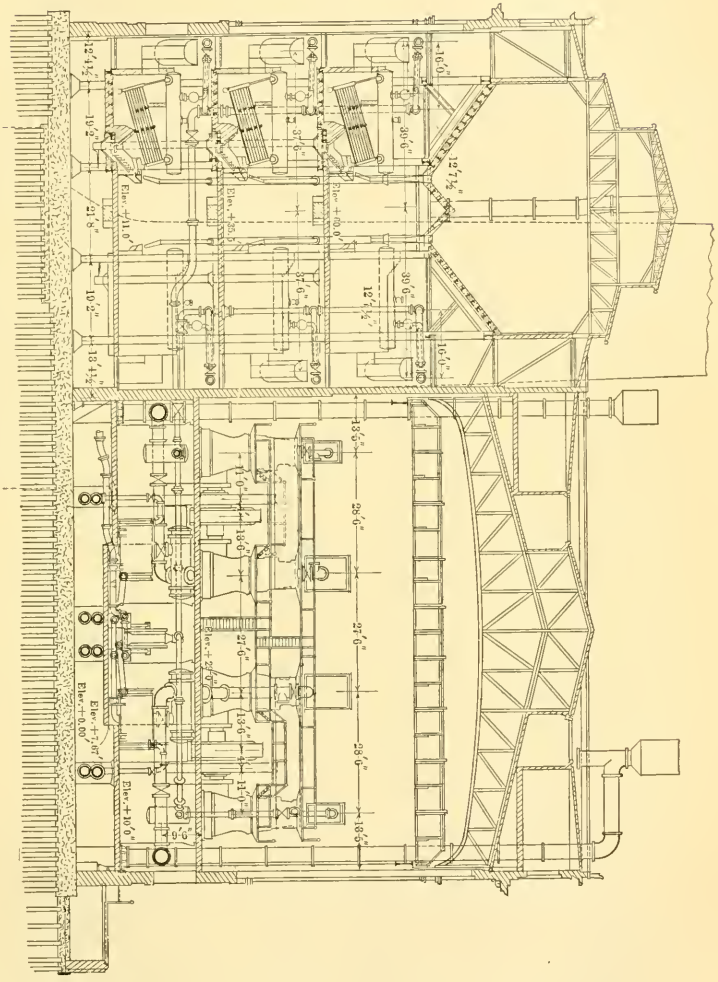
The flues enter from opposite sides on each of the three floors, and a baffle wall in the center, reaching just above the highest flue opening, directs the gases upward and prevents the discharge from one side tending to choke up the opposite flue.

The chimney is surmounted by an iron cap, cast in sections and bolted together (Fig. 6). The whole is topped by a series of lightning rods, which are grounded by copper ribbons on opposite sides.

Fig. 1, Plate X, shows the excavation for the chimney, partly filled with concrete, and also shows some of the concrete for the surrounding foundations. Fig. 1, Plate XI, is a view of the chimney showing the height to which the outer scaffolding was carried.

The Building.—The building is of brick. The outside is faced with iron-speckled brown brick and is trimmed with granite and with brown fire-flashed terra cotta. The window frames are of iron, and the cornice and monitor trimmings are of copper. Fig. 2, Plate XI, is a view of the completed building, from the river end. The inside walls of the engine room are of common brick, but there is a wainscoting of enameled brick up to about 9 ft. above the floor. The walls are for the most part self-supporting. The floors, floor loads, roof, roof loads, boilers, coal pocket and contents are carried by the columns. There is a heavy cornice of terra cotta blocks near the top of the building and this is partially supported and anchored by iron frames, but these are so constructed as to allow a slip down of 1 in. if the wall below should shrink or settle away that much from the iron frame of the building. Other mouldings, cornices, etc., are built into and supported entirely by the walls. Fig. 2, Plate X, shows the ironwork of the building during erection.

The floor loads allowed are as follows: Engine-room floor, 400 lbs. per square foot on the part between the engines, and 600 lbs. in front,



CROSS-SECTION OF POWER STATION LOOKING WEST.

Fig. 4.

which amply allows for the concentrated loads of the exciter engines and exciter switch-board; sub-engine room floor, 225 lbs. between the engines, with special provision for condensers, heaters and pumps; and 600 lbs. in front where are supported the transformers and rotary converters for a sub-station for the power-house locality; 225 lbs. per square foot on each switch-board floor; 175 lbs. on the pent-house floor; 100 lbs. for the roof; 425 lbs. on the boiler-house floors for the part in front of the boilers, and 225 lbs. for the space between and back of the boilers. These figures include the allowance for the floor itself and for the superficial loads.

No provision was made for longitudinal expansion, as it was considered that it would act each way, from the big brick chimney as a fixed point, and take care of itself. The boiler-house joints are mostly bolted at the bottom, so as to absorb some of the expansion, and are solidly riveted at the top. For lateral expansion the engine-house roof trusses are hung on end pins to allow for any slight movement north from the boiler house, which is rigidly held. The engine-house columns are very long, and those on the north side are wide enough and strong enough to resist the vertical load as well as the horizontal outside wind load.

The wind load was taken at 30 lbs. per square foot at the top, and was decreased somewhat toward the bottom. The columns were made strong enough to resist this wind pressure on a span extending from the end truss pin to the ground, as it was anticipated that the engine floor and sub-floor would not be put in until some time after the building was erected. On the south side of the engine room, the columns are of the same outline, but of considerably lighter metal, as they will have no wind load to resist. The engine-room trusses are pin-connected along the bottom chord and riveted at the top. They carry not only the roof and its load, but also the pent house and its load.

The wind pressure from the east or west is resisted in the engine room by a lateral system in every other bay. This starts in the roof and is carried down to the column bases by diagonal rods and lateral braces. The traveling crane girder, engine-room floor girder, etc., are incorporated with this system.

In the boiler house the columns carry not only the floors and the boilers, but also the coal pockets and, in addition, the roof and its load.

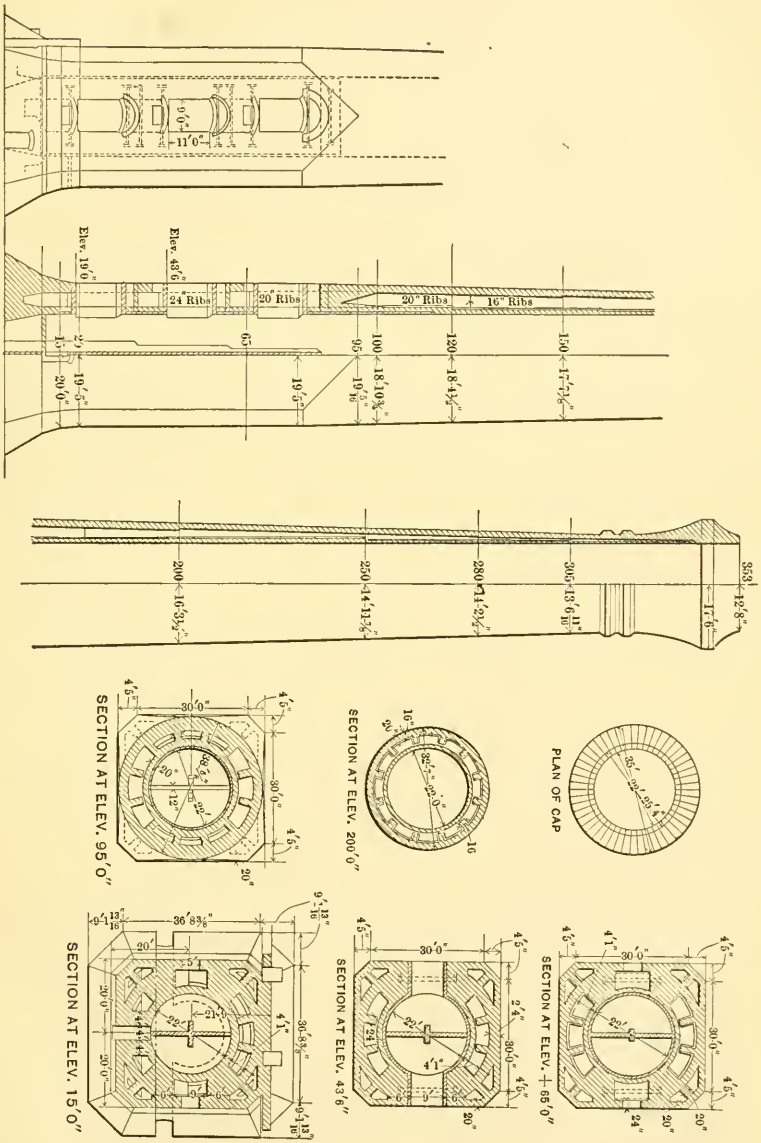

ELEVATION LOOKING EAST
ELEVATION LOOKING NORTH
Elev. 19′0″
Elev. 43′6″
24″ Ribs
20″ Ribs
20″ Ribs
16″ Ribs
SECTION AT ELEV. 95′0″
SECTION AT ELEV. 200′0″
PLAN OF CAP
SECTION AT ELEV. +55′0″
SECTION AT ELEV. 43′6″
SECTION AT ELEV. 15′0″

Fig. 5.

Each pair of columns in the center of a battery is braced diagonally from floor to floor from the coal pocket down.

The large coal pockets presented quite a problem. The tendency to burst or spread out is resisted entirely from the outside, there being no exposed iron whatever in them. The pockets are lined with I-beams spaced 2 ft. or more apart and varying in depth from 10 to 15 ins. These are filled in with concrete which is carried to about 3 ins. over their tops and then covered with a waterproofing material. The concrete surface was left fairly rough in order to prevent the waterproofing from sliding, or peeling off. The section of the pockets is shown on the cross-section of the building, Fig. 4, and the ends are vertical.

The pressures, of course, are calculated for a maximum load, the conditions for which are that the pockets shall hold as much as they can be made to by trimming, that is, that the coal should slope up from the tops of the vertical sides, at the angle of repose, to the summit. The pockets can be filled by the conveyors almost to this limiting line.

The pressures on the vertical surfaces were calculated by Rankine's formula for a retaining wall with superimposed loads:

$$p = wx \cos \phi;$$

in which p = pressure on a unit of surface,

w = the weight of a unit of volume of the material,

x = the depth below the top of this unit of surface,

ϕ = the angle of repose of the material, or the angle of the top slope.

The pressure is parallel to the surface of the material. Then the total pressure on a vertical surface is

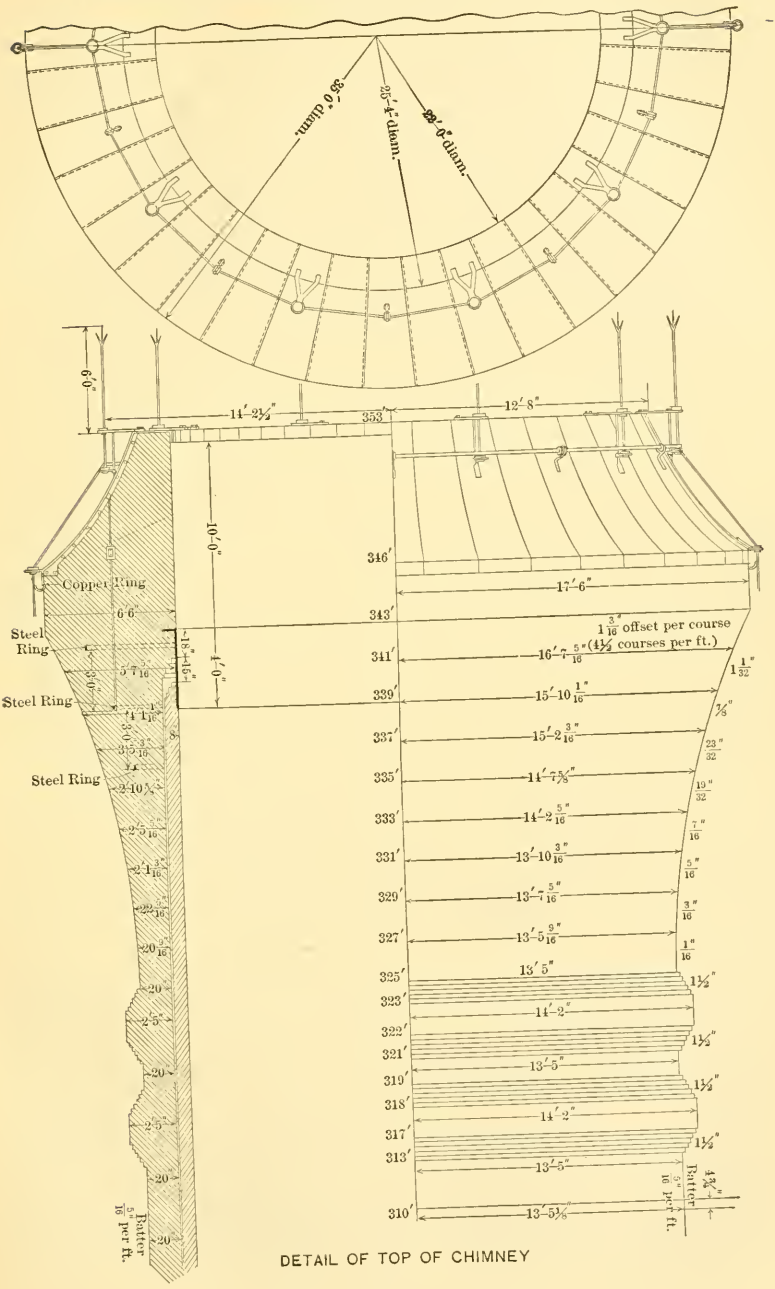
$$P = \left(\frac{d_1^2 - d_2^2}{2}\right) w \cos \phi$$

where d_2 = the depth of the top of the surface, and

d_1 = the depth of the bottom of the surface below that point in the top surface of the material directly over the surface.

The weight of a cubic foot of coal was assumed at 56 lbs.

The pressures on the inclined side portions were obtained by considering each surface as composed of a series of steps, and assuming the pressure on the slope to be the resultant of the vertical dead load multiplied by cos ϕ on the horizontal part of each step, and the thrust on the vertical part as calculated by Rankine's formula.



DETAIL OF TOP OF CHIMNEY

FIG. 6.

The calculated pressures on the vertical faces of these steps were reduced when the so-called line of maximum pressure—that is, the line bisecting the angle between the vertical and the slope of repose—cuts beyond the crest of the coal slopes; that is, intersects the opposite receding slope, or that slope produced, first instead of the rising slope, and by the amount that the part *B C* (Fig. 7) of the line between the opposite receding slope at *B* and the ascending slope at *C* produced, bears to the whole line, up to the ascending slope line produced, or *A C*.

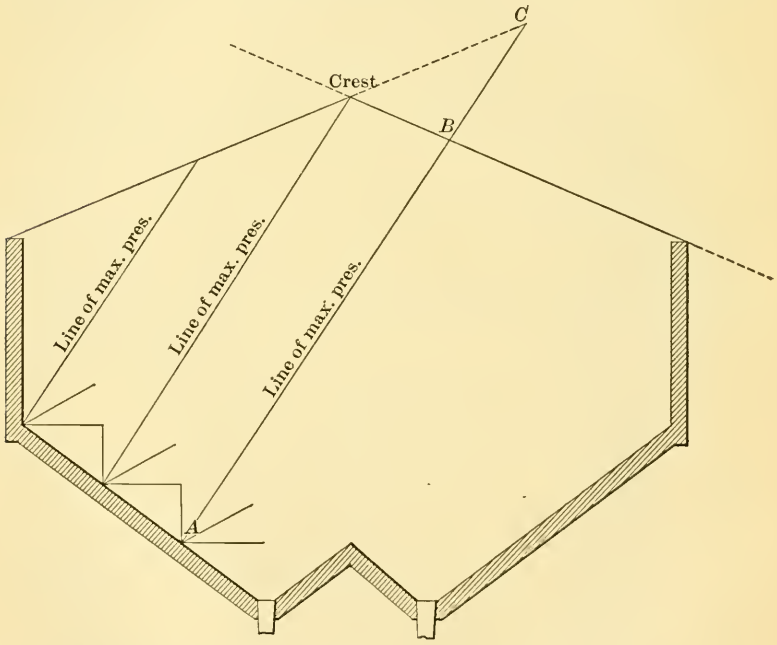


FIG. 7.

In the bottom of the pocket, the two slopes of the crown piece counteracted each other, and therefore it was only necessary to make the V-pieces strong enough to resist this pressure, and the main cross girder only strong enough to bear the vertical load and the tension induced by the side pressures. At the height where the bottom of the vertical portion meets the top of the inclined part there is a horizontal trussing, made strong enough to carry a large portion of the lateral thrust to the corners, and the corners are made strong enough to resist a powerful distorting force.

Across the end of this level there are horizontal trusses strong enough to take the entire end thrust to the corners and thence down the sides where it will be taken up by the frictional resistance of the coal, or, if the pocket is full, it will be partially resisted by the opposite end pressure transmitted along the side pieces.

This was done to provide against any possible bursting tendency of a sudden settlement, one-sided load, or other cause; even for the remote possibility of a slight earthquake.

The strains on the steel, and the loads on the columns and brick work were made to agree with those allowed by the Building Department of New York City by the law used in 1898, before the present law went into effect.

Erection and Cost.—The building excavation was sheet-piled all around because the surrounding material was too soft to take any reasonable slope naturally. The ground-water was easily kept down with one 3-in. and one 6-in. pulsometer, although to do this it was necessary to drive two rows of sheet piling and puddle between them where the cribs running east and west passed out from under the building to the bulkhead. The sheet piling was 3 x 12 ins.

For the chimney foundation a row of 10 x 12-in. tongued and grooved sticks was driven around it in the form of a square. The enclosed space was then excavated to the required depth. The corners were well braced and the piles were driven from the surrounding cellar bottom by attaching to the regular pile drivers sticks corresponding to ordinary derrick booms, and hanging to their ends pairs of ways reaching to the bottom of the chimney excavation. In these ways the hammers were moved and the piles were driven. By this method it was possible to keep several machines at work on the somewhat limited area, and the bother and expense of lowering a machine into the hole and lifting it out again was avoided. The sidewalk vault was excavated and built after the building was nearly finished. The shoring for the earth outside was braced against the building.

Much of the iron-work was erected with house derricks and those of the ordinary form, but the engine-house roof required quite an extensive traveler. At one side it was supported on the tops of the engine foundations and at the other it was at first built on high blocking, and, after two spans (or one bay) of the engine room were up, it

was allowed to run on that side on the main-crane girder, on account of the temporary engine house being in the way.

So that the erection of the engines might proceed before the engine-room roof proper was put on, a temporary roof was suspended from the trusses in the two bays of the engine room over engines Nos. 1 and 2.

The intake pit for the circulating water was made by driving 10 x 12-in. tongued and grooved sticks around on its four sides and using these for its walls, both supporting and retaining.

The roof load is distributed on these sheet piles by framing the roof beams into a heavy beam running continuously along the wall. The sheet piles are cut off so low that, while they are not below tide-water, capillarity will keep them wet and reduce decay to a minimum.

To receive the large engines and generators a runway was built on a level with the engine-room floor from the bulkhead to an opening left in the wall; so that, after a single movement from the lighter to the runway, by means of a floating derrick, the parts could be rolled within reach of the large traveling crane.

Cracks in masonry seem to be unavoidable, and two small ones, running from the flue opening upward for a short distance, developed in the chimney, immediately after it was put in use. These have not increased in size, and there have been no settlement cracks in the building, chimney, or machinery foundations.

The excavation, of which there were about 25 000 cu. yds., cost about $1.20 per yard. About 8 000 piles were used. The 35-ft. piles cost $5.50 apiece, and the 45-ft. piles under the chimney cost $7.50 apiece, driven. The concrete used was a mixture of one part cement, two parts sand and four parts broken stone, and cost $5.00 or $4.50 per cubic yard, according to whether American Portland or Rosendale cement was used. The former was used in the foundation concrete and the latter above Elevation + 0. The chimney, above the elevation where the brickwork commences, cost $20.50 per thousand for everything, including damper ironwork, cap, lightning rods, etc.

About 4 000 tons of iron were used for the building, including architectural cast-iron as well as structural work. The total cost of the station, including machinery, will not be far from $3 000 000.

The foundations were put in during the summer and fall of 1897. In 1898 a temporary engine house, shown in Fig. 2, Plate X, was

FIG. 1.—CHIMNEY EXCAVATION AND CONCRETE FOUNDATIONS.

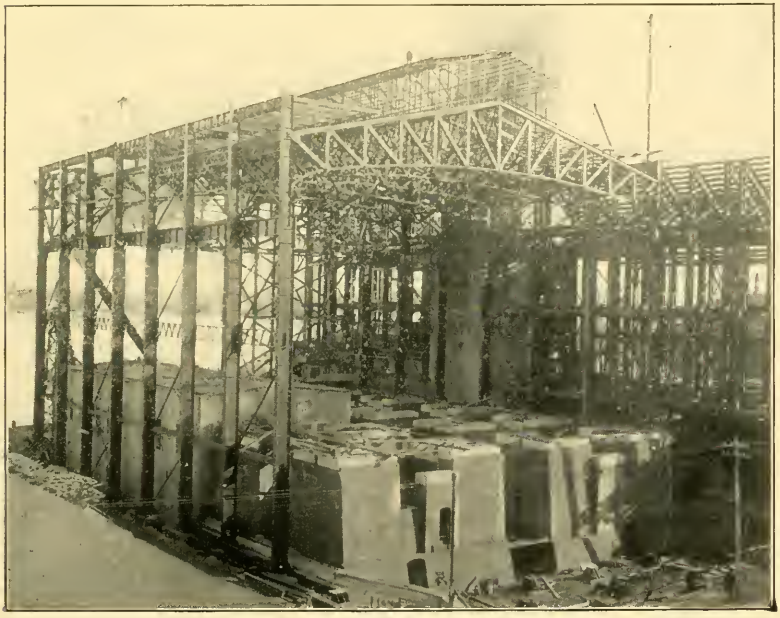


FIG. 2.—FOUNDATIONS OF ENGINES, AND FRAMEWORK OF BUILDING.

built inside the proposed permanent one, and several boilers were erected on the first floor of the boiler house and temporarily closed in. See Fig. 1, Plate XI. These gave an electric power of 1 700 K.-W. for about a year before the permanent plant was put in operation.

The chimney and engine foundations were also put in during 1898. In 1899 the building was erected, and during that year the plant was pretty completely equipped.

Engines, Generators, Etc.—In the engine room there are eleven large vertical cross-compound engines, each directly connected to a three-phase alternating current generator in the center between the cylinders. The engines are piped so that either cylinder may be operated alone, and in that case each is capable of generating 4 500 H.-P., maximum. The engines are each of 4 200 H.-P. normal, and of 6 600 H.-P. maximum, capacity. Each generator has a capacity of 3 500 K.-W., and is capable of carrying a 50% overload for a short time.

For a medium-sized cross-compound engine it is quite customary to ship the shaft, cranks, fly-wheel hub and generator hub in one assembled piece. This was deemed inadvisable in this case, on account of the great size and weight of the pieces and the difficulty of moving them. The pieces were, accordingly, shipped separately, the largest pieces being the shafts, which weighed about 35 tons. Each shaft and the pieces pertaining to it were assembled near their foundation, the different parts being forced on the shaft by means of a hydraulic pressure of over 300 tons.*

The shaft is a large hollow forging of 37 ins. maximum diameter, the hole being 16 ins. in diameter. The cranks are of the style known as fan-tail, in distinction from disc cranks or the ordinary unbalanced cranks.

The cylinders are 46 and 86 ins. in diameter, the stroke is 60 ins. and the number of revolutions is 75 per minute. The fly wheels have a rim speed of 6 600 ft. per minute. They are built up; the hub, arms and central third of the rim are of cast steel, and the outer thirds of steel plates. The plates are in thicknesses of 1 in., no two joints come opposite, and they are fastened to the central part and to each other by long 2-in. steel rivets, which were driven cold by hydraulic pressure.* The fly wheels are very heavy in order to absorb quickly the

* A description of this work can be found in *Power* for August, 1899.

speed variations, for which displacement curves were very carefully calculated.

The valves are of the Reynolds-Corliss type. The valve chambers are cast in the removable heads of each cylinder, and are at the center, a new scheme by Mr. Reynolds, of the Allis Company, by which clearances in the cylinders are reduced.

The connecting rods are of the locomotive pattern, that is, I-shaped instead of round, and the bearings are water-jacketed, as well as piped for the oil supply.

The generators are of the revolving-field type, with fixed outside armatures. The revolving field has 40 poles, and an alternating current of 6 600 volts is generated. The armature was shipped in two pieces and the field in several pieces, as follows: The hub or spider, the rim in four parts, and each pole piece by itself. The armature is of the side-moving type, as made by the General Electric Co., and the pole pieces may be exposed to view without removing the top half of the armature.

The engines stand on the floor at Elevation + 29, and reach about 36 ft. above the floor. Fig. 1, Plate XII, is a view of Engine No. 1, during erection. Fig. 2, Plate XII, shows the interior of the engine room, with eight engines nearly completed.

A separate feed-water heater and condenser is provided for each engine, and is placed in the space under the floor and between the engine foundations. Each condenser has a steam engine driving a single-cylinder circulating water pump and a two-cylinder air pump, the cylinders of which are on either side of and are part of the same frame as the circulating water cylinder. The single steam cylinder is directly over the circulating water cylinder in the center. The exhaust steam from the pumps is so connected that it may be turned into its condenser, into the low-pressure cylinder of its large engine (and thus used expansively), or conducted back into the boiler house to the secondary feed-water heaters.

The exciters are placed at the west end, under the switch-board galleries. Provision has been made here for steam-driven exciters of 60, 30 and 60 H.-P., respectively. The drivers are tandem-compound high-speed steam engines of the "Ideal" make, and are directly connected to small direct-current generators. There is also space for an exciter switch-board and a battery of three electrically-driven exciters, each consisting of a generator coupled to a motor.

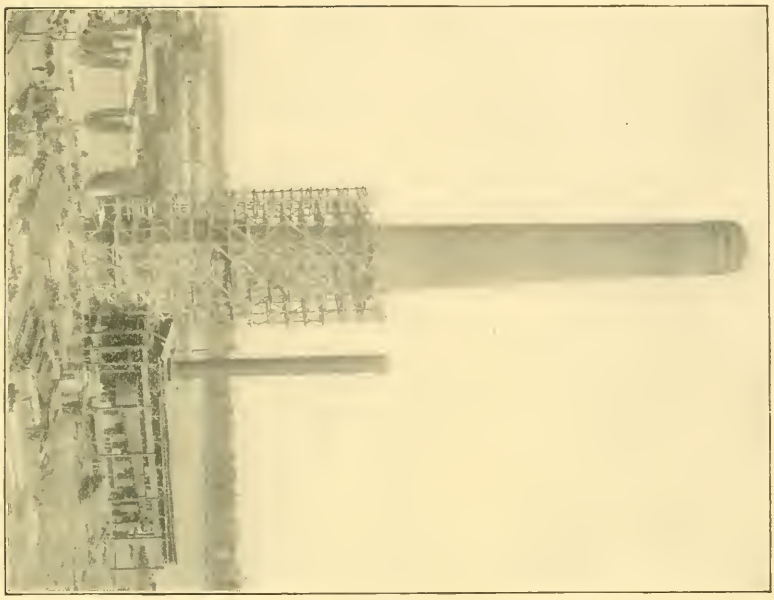


FIG. 1.—CHIMNEY NEARLY COMPLETED, AND TEMPORARY BOILER-HOUSE.

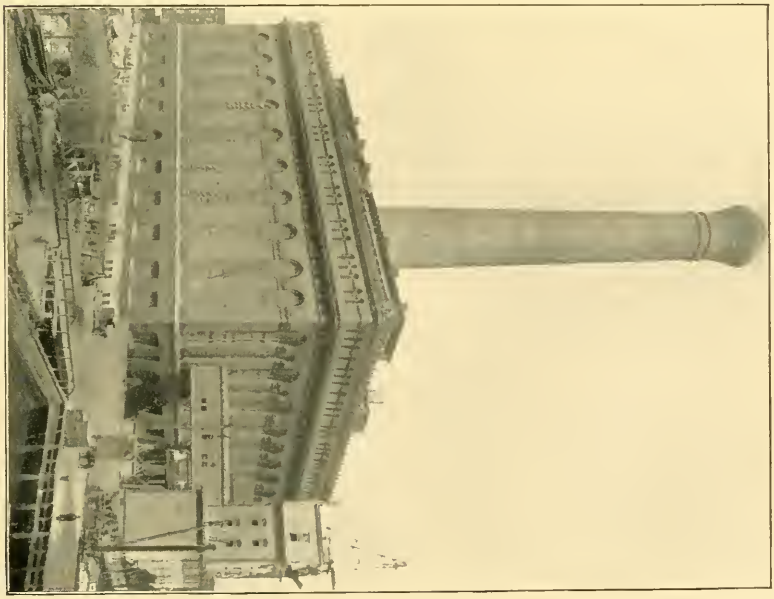


FIG. 2.—VIEW OF THE STATION FROM THE RIVER.

The entire complement of engines has not yet been installed, nor has the third floor of the boiler house been filled. The other engines will be needed if the load of the Third Avenue Railroad is thrown on this station, and the remainder of the third tier of boilers will be filled out with the boilers from the various small electric and cable-driving stations about town, as these stations are discontinued.

The Boilers.—There are 87 boilers, set in three tiers of 29 each. Each boiler is of about 500 H.-P., ordinarily, or 800 H.-P., maximum. These are known as 250 H.-P., ordinarily, when rated by the Philadelphia Centennial Standard, or an evaporation of 30 lbs. of water for 1 H.-P. They are equipped with mechanical stokers, coal chutes to feed them with coal, and ash chutes to convey the ashes to the basement. The stokers are driven through jack-shafts near the tops of the boilers, and the bearings are bracketed out from the columns.

Each line of from 6 to 8 boilers has two small Westinghouse steam engine drivers on platforms between the batteries, the other spaces being planned to be occupied by electric motors. The shafts are propelled by means of link belts. Above the boilers is the coal pocket, divided in half by the chimney, and fed by two link-connected overlapping bucket conveyors, Fig. 8. The total capacity of the double coal pocket is 10 000 tons.

Cranes.—The engine room is provided with several cranes. There is a large three-motor electric crane of 110-ft. span, built by Wm. Sellers & Co., and capable of lifting and carrying 30 tons. An outboard crane feeds the traveler from Exterior Street, and has a hand trolley of 30 tons capacity. A small hand-power crane of 15 tons capacity runs under the first switch-board gallery cross-wise of the building. Another similar one is located directly below it for the sub-station. Provision is also made for a trolley over engine No. 11, and feeding from that to the outboard crane.

Coal and Ash Conveyors.—Coal is unloaded at the wharf by a tower with a fixed horizontal boom, and is transmitted into the pockets by two bucket conveyors. The buckets are 24 x 28 ins., and overlap so as to be continuous-loading without the use of a filler. Sufficient vertical space is provided in the tower for a crusher, if the coal is lumpy, and for two weighing hoppers to be used before the coal is emptied into the conveyors. Each conveyor makes two vertical turns

of 90° and a horizontal twist of about 30 degrees. The length of each conveyor is about 800 ft.

The ashes are picked up from the chutes in the basement and transferred by small trucks on parallel tracks, or by two conveyors, to the river end of the building. Here they are dumped into a vertical bucket conveyor and carried up to the second story where they are again dumped and carried to the ash pocket by the returning coal conveyor. The ash pocket is under the coaling tower, and is about 25 ft. wide and 60 ft. long. It is high enough and of sufficient V-shape to discharge its contents by four chutes into scows at the dock.

Each conveyor is driven by a separate steam hoisting engine. The conveyors are supported from the coaling tower to the building by a steel bridge having a passageway between and on either side of the conveyors. The vertical portions are encased in steel frames, and here the twists are made. The top horizontal parts are supported on the boiler house trusses.

Steam Piping.—The maximum diameter for the mains of the steam pipe system was fixed at 16 ins. on account of the difficulty of placing in position curved lengths of larger size. It is also more difficult to provide expansion for larger sizes and to keep them tight.

At each end of each floor in the boiler house there is a 16-in. loop, and the loops at each end are connected to each other. Each boiler is connected to the line of pipe back of it. Each floor has a connection to each stand-pipe, of which there are as many as there are large engines, 11 in all. These are situated near the loops, in the spaces between the batteries of boilers. From the bottom, and below the level of the engine-room floor, a connection runs across to each engine. Before the steam enters the high-pressure cylinder, it goes through a separator. If necessary, steam can be supplied to any engine from any floor of boilers at either end, although, of course, under normal conditions, steam will be taken from the adjacent part of one of the loops.

Any damaged portion of a steam pipe may be cut out by means of valves in the boiler house without cutting out more than one battery of boilers. A leak or break in the engine room would of necessity cut out one engine. There are various small steam pipes for the smaller engines.

The pipe is all of rolled steel with cast-steel fittings. The flanges made of the latter are in part riveted to the pipe and in part connected by the Van Stone patent.

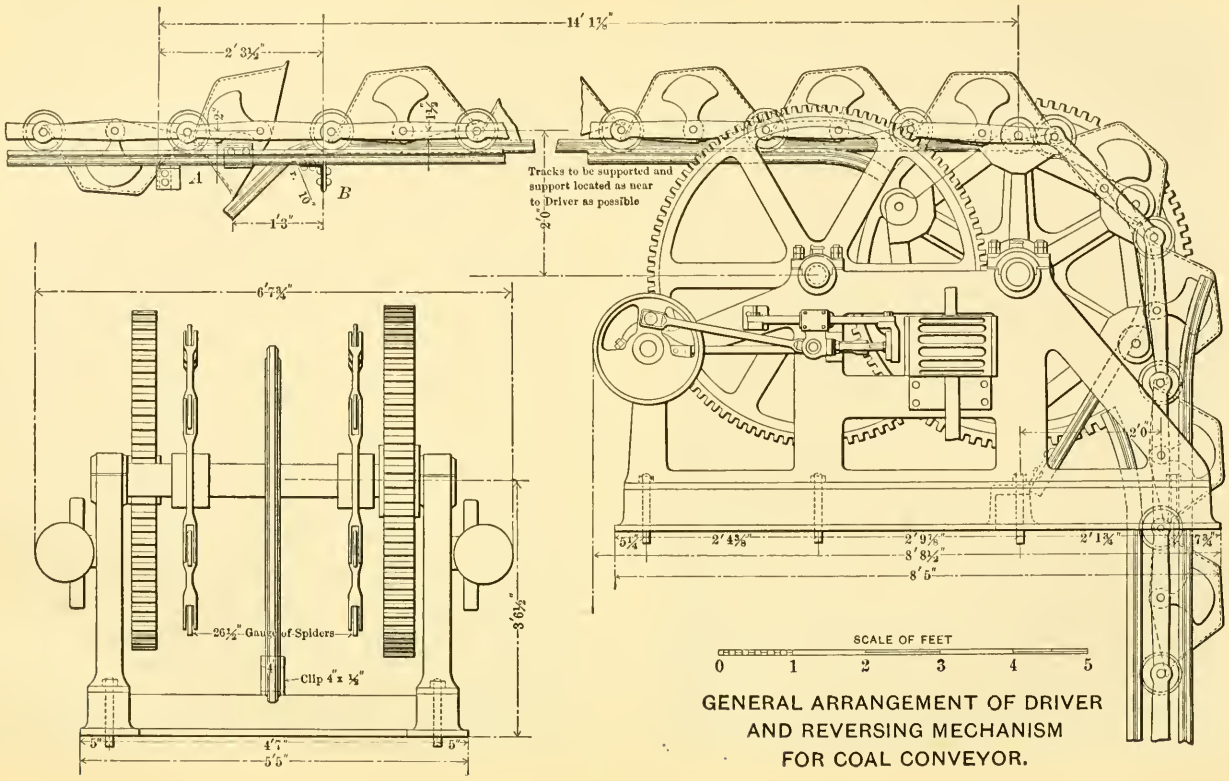

14′ 1⅞″
2′ 3½″
1½″
A
B
10°
1′3″
Tracks to be supported and support located as near to Driver as possible
2′0″
6′7¾″
26½″ Gauge of Spiders
Clip 4″ x ½″
3′6½″
5″
4′7″
5′5″
2′0″
5¼″
2′4⅜″
2′9⅜″
8′8¼″
8′5″
2′1¾″
7¾″
SCALE OF FEET
0 1 2 3 4 5
GENERAL ARRANGEMENT OF DRIVER AND REVERSING MECHANISM FOR COAL CONVEYOR.

Fig. 8.

Exhaust Mains.—Four 42-in. risers are provided for free exhaust, or one for each three engines; and each engine exhaust is provided with a back-pressure valve. In the boiler house there are also smaller exhaust risers as outlets if the exhaust for the feed pumps and auxiliaries is passed through the secondary heaters. The water condensed from each large engine is passed through into its separate hot well where it can settle, become cool and have most of its oily matter removed by a filter at one end. The hot wells are large enough to hold the water for about half a day, if none is passed away and wasted.

Water and Gas Supply.—The water supply is obtained from two separate systems: A main on Second Avenue supplying a section of the city further down town, and a small main supplying the immediate locality. The former connection is on Ninety-fifth Street, is 12 ins. diameter, and is piped to four 6-in. meters. The latter connection is on Ninety-sixth Street, is 6 ins. diameter, and is piped to two 6-in. meters. From the meters the water can go either direct to the feed pumps or to two tanks which may be called equalizing tanks, and in which the water is received from the various sources and mixed or equalized. There is also a 12-in. connection for salt water, which may be used in the feed system if other sources fail, or it may be used for flushing out the basement.

An emergency gas connection will probably be made, so that, if the electric power is absolutely suspended for any reason, the building will not be without light.

Feed-Water.—The feed-water system is provided with three duplex steam-driven plunger-packed compound pumps, of horizontal pattern, and an alternative system consisting of a gang of inspirators capable of elevating a portion of the supply required. The pumps have to work against a pressure of from 160 to 180 lbs.

The feed-water is pumped from the equalizing tanks, or the mains direct, into long mains under the first floor. From these it passes into one or all of the primary heaters in the engine room, or directly into risers which feed through loops into the boilers on the various tiers. From the primary heaters it returns either to the secondary heaters or to the boilers directly. If taken through the secondary heaters it then passes to the boilers. Thus it may take one of the following three courses: To the boilers direct, either at the street temperature, or with that temperature modified by the hot wells; to the

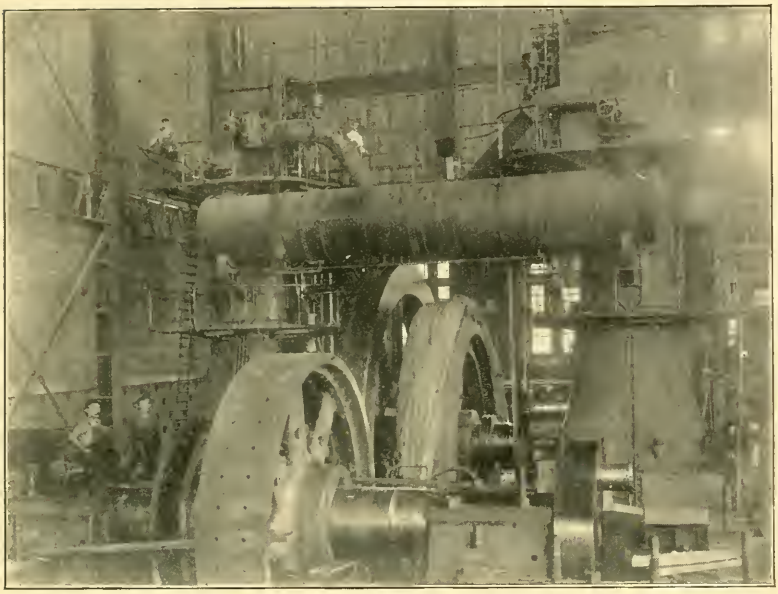


FIG. 1.—ENGINE NO. 1, DURING ERECTION.

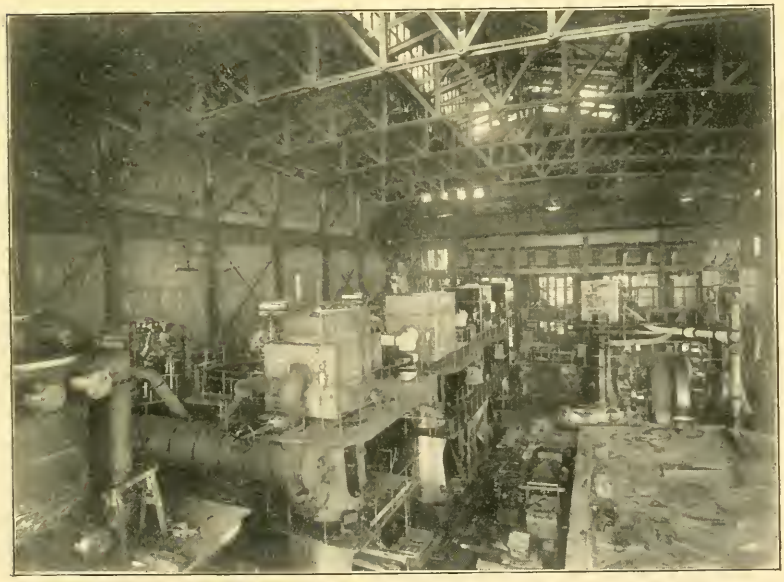


FIG. 2.—THE ENGINE ROOM.

primary heaters and then to the boilers at a temperature of about 125°; or, through both the primary and secondary heaters and then to the boilers at a temperature dependent on the number of secondary heaters installed and the amount of exhaust steam turned into them. If a maximum of four were put in, the temperature would be as high as 200°, or not much below the boiling point.

Circulating Water Piping.—The circulating water system consists of four 30-in. suction mains and four 30-in. discharge mains. Each main supplies three large engines. The circulating water is drawn from a basin or well in the vacant space just inside the bulkhead. It is connected with the river by two 42-in. pipes laid through the bulkhead. The discharge mains go out through the bulkhead and discharge into the river. The water drawn in is taken at a much lower level than that at which the discharge is made. The two masses of water at the widely different temperatures are expected to be kept apart sufficiently by this difference in level.

Oil System.—New oil and old oil from the mains is received at the Exterior Street end of the building, near the entrance and in the basement of the engine room. Three duplex pumps are provided to lift the oil up into the oil room. There it can be passed through one or two filters, or taken directly into one of the receiving tanks in that room. From the oil room, at Elevation + 84, the oil is piped to all parts of the large engines, and also to most of the small engines with sufficient head.

Sub-Stations.—Electric sub-stations have been laid out and several have been installed. As soon as these are finished and the cables are removed, the various cable and direct-current electric stations in the city will be gradually abandoned.

The outfit for the sub-stations consists of several rotary converters, their transformers and a couple of blowers to keep the latter cool.

Contractors.—The main features of the engineering work were by the following firms: Most of the structural work was done by the New Jersey Steel and Iron Co., the masonry by M. Reid & Co., the large engines by the E. P. Allis Co., the generators by the General Electric Co., and the boilers by the Babcock & Wilcox Co.

Staff.—Much of the small piping was done by day's work under the direction of the company's engineers, and many small bits of ironwork were let to various firms in the city. The writer had general

charge of the plans in the office, and made many of the calculations under the supervision of the Chief Draughtsman, C. E. Corby, Assoc. M. Am. Soc. C. E., and the Chief Engineer, M. G. Starrett, M. Am. Soc. C. E. F. S. Pearson, M. Am. Soc. C. E., was Chief Engineer when the building was commenced; and afterward and up to the present time, Consulting Engineer.

Mr. W. A. Low was the Civil Engineer, and Mr. A. S. Mann the Mechanical Engineer, at the building site, in charge of the work.

Mr. A. V. Porter was the Architect of the building, and Albert Carr, Assoc. M. Am. Soc. C. E., at that time the company's Principal Assistant Engineer, looked after the foundation work for the new building. Mr. W. A. Pearson, Chief Electrician for the company, superintended the wiring and switch-boards, and Mr. R. C. Brown was assistant to Mr. Starrett during the construction of part of the work. All the work was done at the office of the company, except the designing of the engines, generators and other machines. General and special features of these were specified. A large portion of the structural iron-work was designed under the supervision of, and checked by, J. T. N. Hoyt, Assoc. M. Am. Soc. C. E., and the remainder, including the coal pocket, by the writer.

Summary.—The station is unique in the following respects, at least: It is the largest steam-power plant in the world, and generates more power per square foot of ground surface than any other. The power generated is 0.94 H.-P. per square foot as an average, or 1.47 H.-P. as a maximum, for the building only; or 0.70 H.-P. per square foot as an average, or 1.10 H.-P. as a maximum, for the whole property, down to the river.

Everything proceeded smoothly, although somewhat slowly, as is frequently the case with such a large piece of work. Many of the schemes adopted may be criticised, but they were selected after a careful consideration of all available methods. Expense has not been spared. It was the rule to obtain the best, but as cheaply as possible. Considerable attention has been paid to the interior and exterior appearance.

The writer is much indebted to the company for the use of the photographs accompanying this paper, and to Mr. Corby for reviewing the same and advising him of the progress of the work.

DISCUSSION.

F. L. Averill, Assoc. M. Am. Soc. C. E. (by letter).—The design of coal pockets for steam plants has, within a few years, received considerable attention, both as to the stresses induced in the framing and in the details of construction. It is to be hoped that the author will incorporate further details of this notable pocket in the final publication. Mr. Averill.

With the present knowledge of the subject, it is surprising to note how recently large steel pockets have been erected without precautions being taken to avoid the certain corrosion. There have even been cases where some of the principal members of the framing have been exposed to contact with the coal.

The disastrous failure of a large pocket in Paterson, N. J., in 1897, created a discussion, among engineers and in the technical papers, of much practical value, which has borne fruit in the more recently designed pockets.

The maximum loading and the prevention of corrosion are now well defined in good practice. There is another point sometimes lost sight of in the design of such pockets: namely, avoiding "dead spaces" between the chutes, where coal can remain lodged. It is asserted by men of experience that any mass of bituminous coal left stationary is certain, sooner or later, to ignite by spontaneous combustion. The writer knows of one large storage pocket where men are sometimes put in to move coal by hand to guard against danger from fire.

The writer recently had to design a coal pocket for the United States Bureau of Engraving and Printing, at Washington, D. C., incident to extension and improvements in the steam plant. There were limiting conditions, partly architectural, so that an ideal arrangement of coal-handling apparatus was impossible. The scheme of framing and the details of construction, however, may be of interest.

The bottom part of the pocket was made in the shape of several inverted pyramids, thus avoiding any spaces for the lodgment of coal, and at the same time permitting the use of deep lattice girders between the several pyramids. These girders give stiff bracing, and support a large proportion of the load.

Expanded metal, wired to angles, was used to prevent the cracking of the concrete.

For the vertical sides of the pocket, brick filling was used between the beams, as one side of the pocket formed the outside wall of the building.

Since reading Mr. Montony's paper the writer has determined to recommend waterproofing in addition to the cement and concrete protection for the pocket at the Bureau of Engraving and Printing.

Mr. Averill. The writer was ably assisted in the foregoing work by Kort Berle, Assoc. M. Am. Soc. C. E., who made most of the calculations and worked out the details.

Mr. Carr. ALBERT CARR, Assoc. M. Am. Soc. C. E.—In regard to the construction of the foundation work there are some minor points which the author might have put a little differently if he had consulted with some one having to do with the execution of the work.

The only point of real importance, perhaps, is the statement in regard to cement and concrete. The author states that "the concrete used was a mixture of one part cement, two parts sand and four parts broken stone, and cost $5 or $4.50 per cubic yard, according to whether American Portland or Rosendale cement was used." So far as the speaker is aware or has any record, not a barrel of Rosendale cement was used in this work, and no concrete was mixed in the proportions quoted.

The concrete under the chimney was nearly all mixed in the proportions, 1 part cement, 3 parts sand and 5 parts broken stone. The concrete outside the chimney was nearly all mixed in the proportion of 1 part cement, 4 parts sand, 4 parts ¾-in. broken stone and 4 parts 1½-in. stone.

The cost quoted is substantially correct, but the difference in cost is due to the difference in the proportions of material used, and not to difference in cement, as stated. Also, a small part of the difference in cost is due to the fact that part of the concrete was machine mixed and part hand mixed.

The author states that the excavation for the building was sheet-piled all around. The excavation was sheeted on three sides complete, and on the fourth side only far enough to cross the cribs which are mentioned. It is an interesting feature of the sheet-piling that the side of the excavation left unsheeted was toward the East River, which was about 60 ft. distant.

The sheet-piling crossing the cribs mentioned consisted only of a single line of 10 x 12-in. tongued-and-grooved timber, and was not puddled. Two lines of sheet-piling were driven across the crib at the north side of the excavation, but the driving of the second line was due to a change in location of the east wall of the building, and not because a single dam was not entirely successful in holding back the water.

Mr. Low has amplified Mr. Montony's description of the sheet-piling around the chimney foundation. The speaker is responsible for the plan of holding back the sheet-piling by tie rods instead of using the ordinary method of bracing inside the foundation, and he was much gratified at its success. Three sides of the excavation were kept practically in straight lines, and the fourth side bulged into the excavation only 6 ins., so that there was no serious movement in the

material back of the sheet-piling. By this method the expensive and troublesome bracing inside the chimney was done away with, and the entire space inside of the excavation was available for the use of pile drivers. All piles were driven without moving the timbers, as would have been necessary with the ordinary method of bracing. Mr. Carr.

The author seems to imply that the plan of using piles under the foundation was adopted because the character of the soil was not known with sufficient definiteness.

Test borings were made, as stated, over the plot to a depth of — 75 ft., but one preliminary boring was carried to rock, which was found at — 123 ft. The original intention was to found the chimney upon rock, and if the depth had not been more than 75 ft. this plan would have been carried out, but it was not deemed feasible to carry the foundation down to — 123.

The borings showed, as stated, filled-in material to the depth of about 15 ft. Below this was a stratum of packed river mud or clay, about 10 ft. thick, and below this were strata of sand beginning with coarse sand on top, and gradually increasing in fineness until about the level of — 30, where fine beach sand was encountered. This stratum of fine sand determined the length of the piles.

Many test piles were driven to refusal, and these indicated that sticks about 45 ft. long could be used. Very few piles, however, over 40 ft. long were driven. The piles were driven to a 1-in. penetration with a 2 500-lb. hammer dropping 20 ft.

In other respects, so far as the speaker is aware, the statements made by the author are correct, and the discrepancies noted are probably due to the fact that plans were sometimes changed in the field after the design had left the Chief Engineer's office.

W. A. Low, Esq. — There was a little settlement of the chimney, but that was anticipated. Careful records were kept from week to week. The speaker took the levels once a week, and found that, as the chimney went up, it gradually settled a little, and to-day the total settlement is under 3 ins., most of that taking place before the chimney was completed. Since the chimney has been completed there has been added the weight of part of the three boiler-room floors and the coal pockets, where the coal is stored. In the settlement, taken on all four corners at the base, a variation of not more than $\frac{1}{8}$ in. is found in any of the four corners compared with any of the others. Mr. Low.

The speaker does not know how much of that settlement was due to shrinkage in the concrete.

The load on each pile under the chimney foundation was between 15 and 16 tons. The speaker does not know the load on the piles under the other part of the building.

Two rows of guide piles were driven down and two 12 x 12-in. guide stringers, 10 ins. apart, were bolted to them to accommodate the 10 x 12-in. sheet piles.

Mr. Low. Along the south side of this dam this method had to be changed, as it was found that one line of guide piles struck the timbers of one of the old cribs, causing them to glance inward. Consequently, a single row was driven and the guide stringers were bolted on either side; then, as the sheet piling progressed, the guide pile immediately in front of it was drawn by means of a lever, and so on until completed.

Owing to the fact that this dam—the inside dimensions of which were 88 ft. by 85 ft. 4 ins.—had to be excavated to — 20 ft., and as the piling had to be carried on along the outside of it, it became necessary to take the utmost precautions to prevent the sides from giving way.

Consequently, anchor rods, 1½ ins. in diameter and 30 ft. long, were secured through the guide stringers at intervals of about 5 ft. by means of washers 8 ins. square and nuts and anchored to 12 x 12-in. timbers laid behind the rows of foundation piles already driven around the outside.

The coal pockets are all waterproof, and there are no places where the coal can lodge, there being a complete system of hills and valleys. Mr. Pearson, the Consulting Engineer for the company, and Mr. Starrett, the Chief Engineer, have both had a good deal of experience in coal-pocket construction, and are, therefore, aware of the damage which can be done by the action of the coal on the iron.

As an instance, the coal pockets in the Kent Avenue power-house, in Brooklyn, may be mentioned. These pockets were lined with sheet iron, which corroded entirely through. The pocket at the Ninety-sixth Street power station was built of concrete, the spacing between the I-beams being filled with concrete and carried out 3 ins. beyond the face of the beams. Then the waterproofing—which was put in by a company whose formula, unfortunately, the speaker does not know—was applied, and it was found that it was of such a consistency that the dust of the coal knitted into it, and, so far, has not caused the least trouble.

The object of the belt conveyor is to take the ashes out when coal is not being hoisted. It runs usually during the night, but is sometimes run during the day, when the coal conveyor is not in operation. It was found to be too expensive to run the coal conveyor to take out the ashes alone, and, consequently, the belt conveyor was put in.

Mr. Boecklin. Werner Boecklin, Jr., Assoc. M. Am. Soc. C. E.—The original plan was, as stated by the author, to handle the ashes by means of the returning coal conveyors, but a modification was found advisable, as the coal conveyors are not always running when it is necessary to remove the ashes, and, as the running of an 800-ft. conveyor, merely to transport them a distance of 75 ft. would not be economical, a Robins Conveying Belt was installed for the purpose.

This conveyor consists, essentially, of two parts—belt and idlers.

Mr. Boecklin.

The belt is 16 ins. wide, reinforced along its middle by a coating of rubber $\frac{3}{16}$ in. thick to resist wear. The upper idlers, supporting the belt and its load, are made up of three 5-in. pulleys, two of which are set at an angle of 35° to the horizontal, thus troughing the belt and materially increasing its capacity. At the head end of this conveyor a circular brush, in contact with the return belt, is operated from the head shaft and throws off any ashes which may adhere to the belt after passing the head pulley.

Ashes are brought by means of small cars from hoppers under the boilers and dumped on the floor at the east end of building, and from here they are shoveled by hand into a long horizontal hopper delivering into a bucket conveyor. This conveyor rises to the level of the bridge, and, running horizontally, delivers upon the belt.

Mr. Kent.

J. D. Kent, Esq.—Two engines are used to drive the coal conveyors, and they are between 15 and 20 H.-P., but the speaker does not think they have yet been carded. The smaller engine, when running the belt conveyor, exerts about 6 H.-P. The wet ashes, as they come from the ash pan, do not cause any trouble, but the soot, taken from the back of the boilers, sticks or cakes on the belt and has to be cleaned off. The large buckets are affected in the same way.

Mr. Montony.

L. G. Montony, Assoc. M. Am. Soc. C. E. (by letter).—Mr. Averill expresses the hope that more details of the coal-pocket will be incorporated in the final discussion, but there does not seem to be much more detail to be described, unless the stress diagrams are submitted, and this seems hardly worth while. The point regarding the avoidance of dead spaces where the coal might lodge has been answered by Mr. Low. The cross-section of the pocket shows the bottom to be self-draining in that direction, while the longitudinal section indicates a horizontal bottom; but this is due to the fact that the section was taken on the center line of the boiler-house and not through the coal-chute throats. Between these, the bottom was lifted up in a series of concrete ridges so that the coal would slide clear in one direction if not in the other.

It might be well to state that at one time it was proposed to place a water-pipe around the top edge of the coal-pocket, with numerous hose connections, so that the pocket might be flooded at short notice. The writer does not know whether this is still contemplated, has been done, or has been given up.

The writer is very much obliged to Mr. Carr for correcting any misstatements in reference to the proportions of the concrete used, and regarding the sheet piling. The statement in the paper that "the former (Portland cement) was used in the foundation concrete and the latter (Rosendale cement) above Elevation + 0" should have been "the former was used in the foundation concrete and the latter in the building walls, floors, etc., above Elevation + 5."

Mr. Montony. There was no intention of implying any doubt as to the nature of the earth at the site, down to such a depth as to make piling the only feasible foundation. If there had not been a sufficient number of borings down into the blue clay he would hardly have tried to give as much detail as he did in regard to the various strata and their apparent levelness. While the reports indicated that one boring was carried down to —123 ft. and met a stony or rocky resistance, the writer does not believe this to be at all conclusive evidence that bed-rock could have been found at that level, as the reports indicated that the other borings were not carried much below — 80 ft.

As the conveyors have been put in operation and certain changes made in the proposed method of handling the ashes since the writer has seen the plant, he does not feel qualified to join in the discussion of that feature.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

Note.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

No. 882.

IRRIGATION STUDIES.

By Elwood Mead, M. Am. Soc. C. E.

Presented September 5th, 1900.

WITH DISCUSSION.

The Office of Experiment Stations of the United States Department of Agriculture is now engaged in a comprehensive study of the methods of distributing and using water in irrigation. The results of these studies for 1899 have been compiled, and will shortly be published by the Department. Some of the methods used in these investigations, and the results of the measurements to determine the losses from seepage and evaporation in canals and the duty of water utilized have been extracted from the report, and are presented in this paper.

The investigations described deal with problems which sorely perplex the irrigators and canal builders of the arid West. Their comprehensive study is a new feature of national aid to irrigation development in this country. Heretofore, the leading object of such aid has been to promote the construction of new canals, to show how much land there was above existing ditches which could be reclaimed, and the benefits which would come from such reclamation. It is believed that this investigation will tend to secure these ends, but its primary

purpose is to promote the welfare of the people living below the ditches already built, to render the farms now irrigated more profitable, to lessen the controversies over the distribution of water and secure its more systematic and economical use.

It is the opinion of those best informed that a better understanding of the existing situation must be had before we can wisely plan for future development. Controversies over the use of streams should be ended before an attempt is made to greatly augment such use. The claims to water, for existing and prospective ditches on many streams, amount in the aggregate to many times the supply. The character and extent of the rights now vested must determine what is to be done in the future. In the pioneer stages of western irrigation, the importance of these questions was lost sight of. The owners of the earlier ditches from a river did not need to consider how much water was used or how much was wasted, since neither had as yet made any serious reduction in the supply. Every transaction which had to do with the disposal of streams was marked by a lavish prodigality. Ditches diverted more water than was used; their owners claimed more than they could divert, while decrees gave appropriators titles to more water than ditches could carry and many times what the highest flood could supply. Little was known of the quantity of water needed to irrigate an acre of land, and in the absence of such information the ignorance and greed of the speculative appropriator had its opportunity.

The contracts which control the distribution of water from canals were framed by people to whom the whole subject of irrigation was strange and new. It often happens, therefore, that they do not promote the best interests of canal companies or meet the necessities of users. The laws which govern appropriations of water from streams have in most cases no relation to the actual practice of irrigators, and, therefore, fail to secure either the systematic distribution or best use of the available supply.

Before the period of crude structures and still cruder ideas had ended, it began to be manifest that the reclamation of arid lands involved more than the overcoming of physical obstacles. It has been found easier to dig ditches than to distribute the water they carry, and to plan headgates and flumes than to frame just laws for establishing titles to water or dividing rivers among rival claimants.

The many thousands of miles of canals and laterals in the irrigated regions of the United States have reclaimed an area approximately as great as the State of New York, every acre and almost every square foot of which has to be artificially moistened from one to ten times each year. During the growing season this requires the services of an army of men to protect and regulate headgates, patrol the banks of canals and adjust the measuring boxes of users. The success or failure of these canals is a matter of more than local interest. Much of the money expended in their construction came from the East. The small savings of thousands of thrifty New England people have been invested in stocks and bonds of irrigation companies, a single agency in Colorado having invested $15 000 000 in this class of securities for these customers. The failure of a canal company to find customers for the water carried, or to obtain water for the customers it has, affects many others besides the water user. His is the immediate loss, but sooner or later the holder of the stocks or bonds of the canal company finds that he, too, is suffering through delayed or defaulted interest payments. The justice and efficiency with which the stream is divided, and the economy with which the water is used, may augment or reduce the incomes of many thousands of eastern as well as western homes.

The Distribution of Water Among Users.

Traffic in water is carried on under many peculiar and perplexing conditions. No matter from what source the supply is received, whether it is stored in reservoirs, pumped from wells or taken from rivers, the distribution of water in irrigation is subject to unending uncertainties. Streams rise and fall with every passing cloud. The torrent of to-day may be a dry channel a month hence. Wells which cannot be exhausted in April are often empty in June. Even after water has passed the headgate and is safe from outside interference, the waste and loss continue. It disappears through the bottom of the canal by seepage, and into the air by evaporation. The same vicissitude attends its use. As much water may escape from the lower side of the field of a careless irrigator as sinks into the soil. The waste from badly built laterals or poorly prepared fields does much to limit the acres which a canal can serve, and hence the income it can be made to yield.

This commerce in water has been created by men born and reared in regions of ample rainfall, and without prior training or experience in dealing with the problems of irrigation. They had to learn by trial how to frame satisfactory contracts for the disposal of water from canals, and how to use that water properly when delivered. From the construction of the first small furrows in Utah and California up to the present, the growth in acres irrigated has been accompanied by an equally important evolution in methods. The fixing of a unit of measure to be used in delivering water to users will serve to illustrate this. It could not be sold by the pound or by the ton, nor were there any devices at hand for its measurement or delivery by the gallon. Farmers were at a loss to know how much to buy, and canal companies as ignorant of how much they could sell or how to measure it when sold.

The Units of Volume Used in Measuring Water.

The "Inch."—In a number of the arid states placer mining was an important industry before irrigation began. Miners, in measuring water, used the "inch." This is the volume which will flow through an inch-square orifice under a uniform and designated pressure. Later, the pressure to be used, and the manner in which the size of the orifice was to be increased or diminished, was, in a number of states, fixed by law.

"Water sold by the inch by any individual or corporation shall be measured as follows, to wit: Every inch shall be considered equal to an inch-square orifice under a 5-in. pressure, and a 5-in. pressure shall be from the top of the orifice of the box put into the banks of the ditch to the surface of the water; said boxes or any slot or aperture through which such water shall be measured shall in all cases be 6 ins., perpendicular, inside measurement, except boxes delivering less than 12 ins., which may be square, with or without slides; all slides for the same shall move horizontally, and not otherwise; and said box put into the banks of ditch shall have a descending grade from the water in ditch of not less than one-eighth of an inch to the foot."*

In those sections where irrigation succeeded this form of mining, irrigators generally adopted this unit. In many respects it is entirely satisfactory. Where the flow is controlled by a device of reasonable accuracy it is a convenient method of delivery for canal companies, and satisfactory to users, because they can tell at a glance whether or

* General Statutes of Colorado, Sec. 3472.

not the quantity contracted for is being delivered. It is not suited, however, to the measurement of rivers or to the regulation of their division among large canals, as the prescribed conditions cannot be produced on a large stream of water. There are canals which carry 125 000 ins. To measure this volume, under the conditions prescribed in the Colorado statute, would be practically impossible. The use of the term "inch" has been unfortunate. Many farmers have confused this unit with the square inch or the cubic inch, and it frequently happens that the inches of water bought or sold are determined by measuring the cross-section of a ditch or lateral, paying no attention whatever to either grade or velocity. In one case a state law confuses cubic inches with the continuous flow from an inch-square orifice.

The Cubic Foot per Second.—When it became necessary to gauge streams and to measure the volume of large canals, it was manifest that some other unit of measurement that the miner's "inch" had to be used. The cubic foot per second is the unit which has passed into general use. This unit has the double advantage of showing precisely what is meant, and being well adapted to the measurement of large as well as small volumes of flowing water. It is the most satisfactory unit which can be used in dividing rivers or in measuring the flow of large canals where the flow is continuous. There is, however, an objection to its universal use in water-right contracts, or in decrees establishing rights to water. Where decrees or contracts provide for the measurement of the quantity received as a continuous flow, it presupposes that irrigators use water in this manner. This is not in accord with the best practice. Irrigators do not need water all the time. Few use it half the time. If they are required to pay for a continuous flow, they usually pay for something they do not get, and always for what they do not need. If they are allowed, as an equivalent of a continuous flow, to take a larger volume for a shorter time, a different unit of measurement is desirable; because it is not a stream of a particular size, but the total volume received, which is paid for.

The Acre-Foot.—The growing recognition of the fact that a continuous flow of water does not correspond to the needs of irrigators has recently brought into use another unit of volume, the acre-foot. It contains 43 560 cu. ft., or enough to cover an acre 1 ft. deep. It is a convenient unit for selling stored water, because it can be used to measure the capacity of reservoirs.

Contracts in which the acre-foot is used provide for the delivery of water on the demand of the irrigator, or at intervals rather than in continuous flow; and canal companies have hesitated about adopting this unit, because of a fear that satisfactory arrangements for delivery could not be made, and that more water would be called for at some times than the canal could supply, while at other times the entire volume would run to waste.

Wherever the acre-foot has been adopted it has proven acceptable to irrigators, because they share in the benefit resulting from care and skill in distribution.

Form of Water Contracts and Benefits of Rotation.

Contracts for supplying water take many forms. In some cases they are deeds to the water delivered; in others they purport to transfer a perpetual right to a specified quantity or to a stream of a specified size; in others they agree to provide water for the irrigation of a specified number of acres for one year; while a few provide for payment for the quantity actually used, fixing a maximum quantity which may be demanded in one season. At first these contracts were largely governed by the ideas of the managers of the canals, but sufficient experience has now been had to make certain forms of contracts preferred in behalf of public as well as private interests.

Contracts which provide for the delivery of a uniform, constant flow are, as a rule, wasteful of water, and are not in the interests of either ditch companies or the public. Contracts which charge for the acres irrigated, without regard to the volume used on these acres, are a temptation to extravagance on the part of the irrigator. The canal company which adopts such contracts resembles the grocer who would agree to supply his customers with a year's provisions at so much per head, with no restrictions as to quantity or kind of goods which might be called for. On the other hand, contracts providing payment proportioned to the quantity delivered, and for delivery in amounts which can be most efficiently distributed, cannot fail to lead to economy in the use of water, and consequently to a high duty. Under such a system the irrigator is benefited by his saving, and pays for his waste. Such contracts can only be used in connection with a system of rotation in delivery to irrigators. This rotation benefits the canal company as well as the irrigator, because it lessens the loss

from evaporation and seepage. A canal 60 miles long could be divided into three sections of 20 miles each, and all the loss from seepage and evaporation on the lower 40 miles saved while the irrigators of the upper section were being supplied. In the same way, by keeping the full supply in the canal, water could be rushed through to users under the lower section with less loss than where the flow is depleted by laterals along the route. The greatest saving in rotation, however, would be made in the laterals. Where water is permitted to slowly dribble through continuously, the waste is enormous. By devising a system for grouping the laterals and inducing the irrigators therefrom to take water by turns, the engineer can do as much toward raising the duty obtained as the actual cultivator. The use of a unit which favors rotation between users leads also to rotation in the division of the river between the canals. The loss from seepage and evaporation is approximately the same whether the canals are full or only half full. When rivers are low, by running half the canals at a time with a full supply, nearly half the water ordinarily lost in transit is saved. As the loss from seepage and evaporation averages about 30% of the water flowing in canals, the water saved by such rotation is a material addition to the available supply.

Reasons for Investigation of the Duty of Water.

As the water required to irrigate 1 acre of land should be the basis for fixing the dimensions of works required to irrigate any number of acres, there is need to know approximately its amount. In order to plan for the just distribution of the volume entering the headgate, the losses in transit must be provided for. Until more is known than is now known about the time of year when the irrigation season begins and ends, the part of the discharge which must run to waste unless stored cannot be estimated. Until it is known how large an area an acre-foot of stored water will irrigate, and the returns which will come from such irrigation, the value of reservoirs will have no more substantial basis than individual judgment or conjecture, and no intelligent estimate can be made of the amount of money which can be profitably spent in their construction. Sooner or later, a knowledge of the duty of water becomes a necessity in any irrigated district. It is now urgently needed to settle disputes over water-right contracts, and to provide for their intelligent reconstruc-

tion. Thus far, it has been the uniform practice to make all rights to water perpetual and continuous. This is not the practice of European countries. Italy, France and Spain, each distinguishes clearly between rights to the summer and to the winter flow, the vernal and autumnal equinoxes being the dates when one begins and the other ends. The controversies which have recently arisen over rights to the winter flow of streams will doubtless soon lead to a similar distinction in western irrigation laws. A comparison of the duties secured under many of the canals where measurements were made last year leads to the belief that it will be possible through improved methods to double the average duty now obtained, so that the quantity now required for one acre will serve to irrigate two. If this can be accomplished it will relieve the scarcity under many canals, put an end to many controversies growing out of the existing scarcity, lessen the expense per acre for water, and increase immensely the productive and taxable resources of the arid States.

Believing that a more general understanding of the causes which increase or diminish the duty of water is one of the most urgent needs of irrigated agriculture, the determination of this duty was made a leading subject of the investigation.

Methods Used in the Investigation.

In carrying out this investigation laboratory methods will not answer; it must deal with the use of water on a large scale. The work requires the supervision of men of special training and wide practical experience. One of the chief difficulties encountered at the outset was to find the right men to take charge. Those engaged are, without exception, holding positions of responsibility and receiving ample compensation from other sources. The chief inducement for their taking part in this investigation has been the promotion of the public welfare. Through their interest and zeal a large amount of information was obtained which could not otherwise have been secured for ten times the actual outlay. It was left for the observers in each State to secure the co-operation of intelligent, practical farmers, and to arrange with them to measure the water used on their fields. In nearly every case this was easily accomplished. Every farmer connected with the investigation received the same instruction. It was, to use water whenever and wherever it was thought necessary, pro-

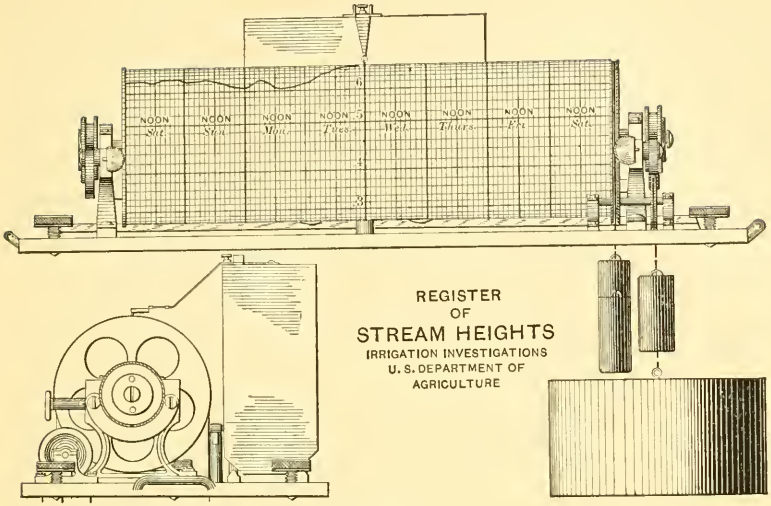
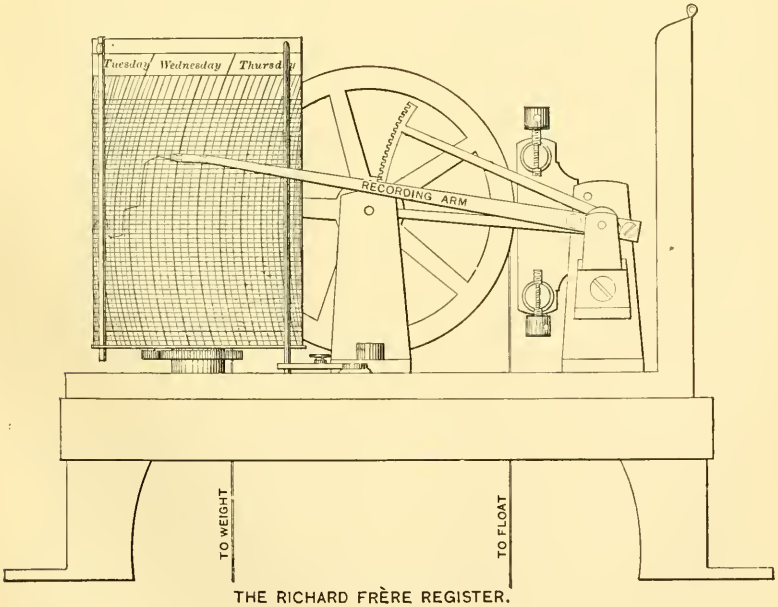


FIG. 1.



THE RICHARD FRÈRE REGISTER.

FIG. 2.

vided it could be had, and pay no attention to the fact that it was being measured. The results show that this was done. The description of canal systems and the methods which govern their operation, given in the reports of the special agents, show how direct is the relation between good management and a high duty of water. They also show how prolific of waste and loss is a badly drawn water-right contract. Records were also kept of rainfall and evaporation, and an effort was made in each case to secure as much information as possible on the following factors of the duty of water in irrigation:

The quantity of water required by different crops.

The length of the irrigation period in different sections of the arid region.

The agreement or divergence between the quantity of water used in irrigation in the different months of the growing season, and the rise and fall of streams during those months.

The benefits of reservoirs, and the percentage of the total discharge of streams which must be stored in order to utilize it all.

Losses in canals from seepage and evaporation.

Influence of different forms of water-right contracts in promoting economy or waste.

The returns from the use in irrigation of an acre-foot of water.

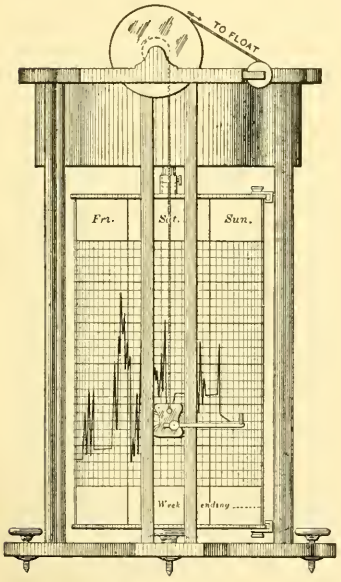
Instruments Used in Recording Amounts of Water Used.

In the study of the duty of water, and to record the quantity used, provision had to be made for an instrument which would be automatic and continuous in its operation. This was necessary because the quantity received by each irrigator from his lateral is subject to frequent change. Finding it impossible to use a meter to measure the volume delivered, it was decided to place a weir or flume in each canal or lateral, and then, by means of a suitable instrument, keep a continuous record of the depth of water delivered. Wherever possible, weirs were used, but in more than half the cases where they were used the results proved that flumes would have been more satisfactory.

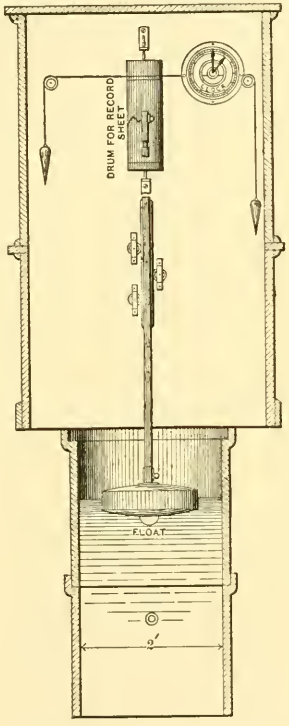
A study of the registers in use showed that none was wholly satisfactory. The first requisite was a record on a natural scale, so that an inch rise or fall in the ditch would be so shown on the record sheets, thus enabling any farmer or ditch rider to determine at a glance

whether or not the instrument was working accurately, and, if not, to correct it without the computation required where the scale is reduced. Not being able to obtain registers of this pattern, one was designed. Its form is shown in Fig. 1.

In this instrument the rise and fall of the water in a ditch or lateral raises and lowers a float and counter-weight. This float and counter-weight are connected by a cord which passes over the end of a cylinder



THE WYOMING NILOMETER
FIG. 3.



REGISTER OF HEIGHT OF WATER
GAGE CANAL, RIVERSIDE, CALIFORNIA
FIG. 4.

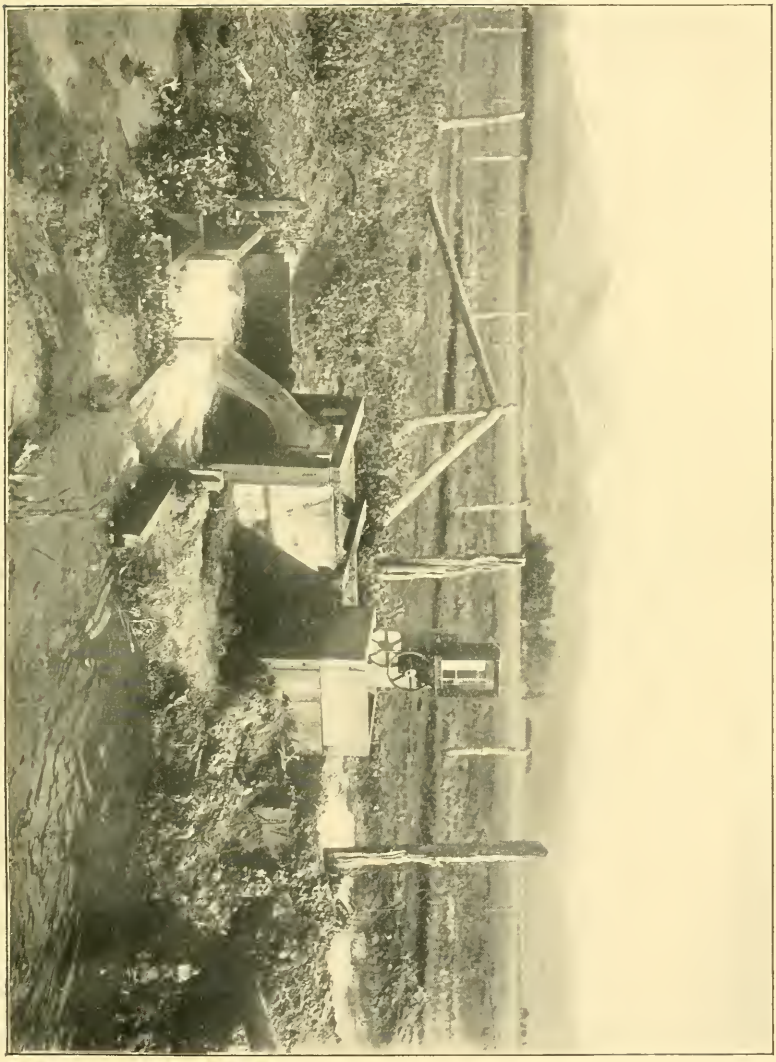
which is revolved by the movement of the cord as the float rises and falls with the changes of depth in the stream. This cylinder carries a paper divided into rectangular spaces, the time divisions being parallel to its axis and the depth divisions at right angles thereto. The pen or pencil making the record is moved along this cylinder by clockwork, and passes from one end to the other in a week, when the paper is

changed and the pen returned to the starting point. The only result of a wide variation in stream depth is an increase in the number of revolutions of the cylinder.

Owing to delay in the construction of these instruments, all observers could not be supplied with them, and a number of registers of other patterns were used. The Richard Frère register, Fig. 2, was utilized in the measurements in Arizona, the instruments being loaned by the University of Arizona. The Wyoming Nilometer, Fig. 3, was used at the Wyoming and Nebraska Stations. The Irving register, Fig. 4, was used on the Gage Canal in California. Plate XIII shows the method of placing registers used in weir measurements.

Relative Merits of Weirs and Flumes in the Measurement of Water.

Reference has already been made to the fact that some of the weirs did not prove satisfactory. This was due to the deposit of silt above them. Sediment investigations made during the season showed that certain southern streams carry, during floods, as high as 5% of solid matter in suspension, and that canals and laterals taking water from these streams have to be cleaned from two to three times each year. Even where the percentage was much less than this, the deposit of sediment was so rapid in some cases as to fill the lateral or ditch above the weir to a level with its crest in 24 hours. Where this happened, the velocity of approach became a disturbing factor, the influence of which could not be determined owing to the constant change of conditions. Some canal companies which use weirs, operate, in connection therewith, a sluicing device which removes the accumulated sediment once each day. The objection to this is that the conditions are never stable, and it is impossible to tell for what length of time the weir tables used agree with the actual discharge. The recent investigations in the flow of water over dams and over weirs, other than those with sharp edges, may aid in securing the adoption of a form of weir better suited to the sediment-laden waters of the Southwest than that used, but, so far as knife-edged weirs are concerned, there are few ditches in the Southwest where it is not possible to secure rating tables for flumes which will give much more reliable and accurate results. It was also found that, in the case of a number of canals, the grades were too small and the banks too low to secure the requisite fall



REGISTER FOR WEIR MEASUREMENTS.

below the weir, and in such cases flumes would not only be preferable but an inevitable substitute.

The most serious objection to the use of flumes is the labor of preparing an accurate rating table, and the fact that a current meter is required for doing this. The recent improvements in these instruments, by which both their convenience and accuracy have been increased, has made it a simple matter to prepare a discharge table for flumes in which the flow is reasonably uniform. With ordinary care this discharge can be determined within the limits of accuracy permitted by the meter used, and in the best instruments this error is as low as 1 per cent. This margin of error is below what is permissible in the delivery of water or attainable in this investigation. The relation of the volume discharged to the depth of water in the measuring

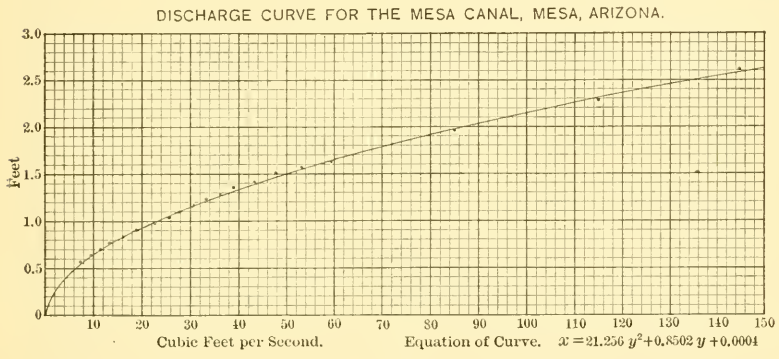


FIG. 5.

flume of the Mesa Canal, Arizona, is shown in Fig. 5. The dots show the gaugings. The curve is plotted from the equation $x = 21.256\,y^2 + 0.8502\,y + 0.0004$, which is derived from the actual gaugings by the method of least squares.

THE UNIT OF MEASUREMENT USED.

In the tables which follow, the acre-foot is the unit of volume used. This is usually expressed by giving its equivalent in the depth to which the water used would have covered the surface irrigated. This unit was chosen because it is definite, and because it affords a convenient base for the comparison of quantities used in localities where the period of use was not the same.

It is usual in discussions of the duty of water to take the cubic foot

per second as the unit of quantity, and the period during which a crop requires water to bring it to maturity as the time during which the flow of that volume continues. Thus, when it is said that the duty of water is 60 or 80 acres to the cubic foot per second, the statement implies that the quantity of water used during the season had been measured and the average volume used during this season amounted to 1 cu. ft. per second for each 60 or 80 acres irrigated. In order to make this expression definite, it is necessary that the duration of the irrigation period be known; but this varies so widely in different localities, and in actual practice in the same locality, that it is difficult to compare the results obtained. In a number of discussions of this subject the assumed season for the Rocky Mountain regions has been taken as varying from 100 to 150 days. The records kept last year show, however, that water was used from the Gage Canal at Riverside, Cal., throughout the entire year, while the canal at Wheatland, Wyo., was operated only 60 days, and water was used in irrigation a shorter time. Since the practice varies so widely, any attempt at fixing an average period would be wholly arbitrary. Even under the same canal the length of the season has to be assumed, because no two irrigators use water for the same length of time. The length of the irrigating period at the several stations is shown graphically in Fig. 6. Nor does the assumption of a continuous flow accord with practice, because on many canals a system of rotation in the delivery of water is already in operation, and even where the contracts provide for a constant delivery it seldom happens that irrigators use water in this way. When, therefore, in practice, 20 miner's or statutory inches are used on an acre for a day, and none at all for the next twenty days, it is an error to discuss the duty as though a single inch had been used all the time. In nearly all the northern States fully three times as much water is used in July as in August. Hence, a discussion which deals with the delivery of water as though the use was uniform during this period is likely to lead to serious mistakes in practice.

By the use of the acre-foot as the unit of quantity all the arbitrary assumptions involved in the use of either the "inch" or the cubic foot per second are avoided, and its use is equally correct and convenient whether the supply comes from streams, wells or reservoirs, whether it is continuous or intermittent, and whether it ends in two months or extends throughout the entire twelve. The flow of 1 cu. ft. per second

DIAGRAM SHOWING DURATION OF IRRIGATION PERIOD
ON MAIN CANALS INCLUDED IN INVESTIGATIONS.

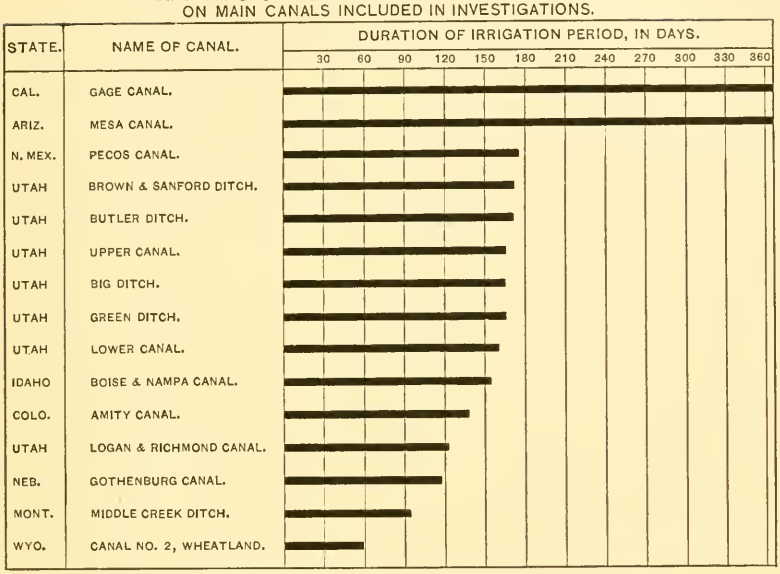


DIAGRAM SHOWING DURATION OF IRRIGATION ON FARMS
WHERE WATER WAS MEASURED IN 1899.

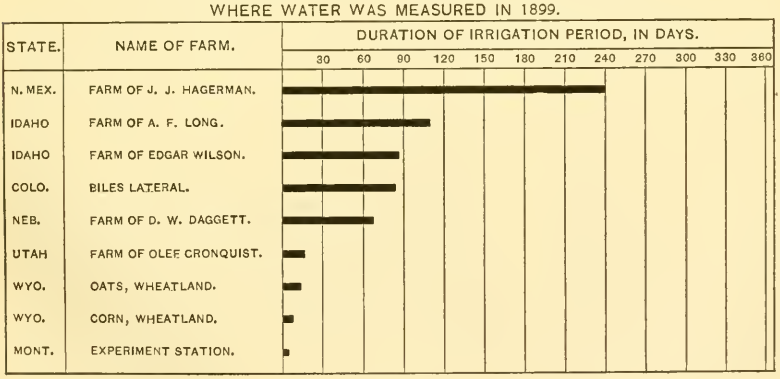


FIG. 6.

for 24 hours amounts to 1.98 acre-feet, so that the conversion of volumes from one unit to the other can be readily made.

Summary of Measurements of Duty of Water.

The measurements made by observers show that the duties obtained vary from less than 1 acre-foot of water per acre irrigated to over 15 acre-feet on an acre; but these wide and seemingly eccentric variations in the quantities used were the results of manifest causes. Where water was distributed through well-built ditches and used by careful irrigators, there was a surprisingly close agreement in results, even in widely separated localities. Table No. 1 will serve to illustrate this.

TABLE No. 1.—Quantity of Water Used, where Measurements Were Made on Small Canals or Short Laterals.

Location.	Depth of water used, in feet.
Cronquist Farm, Utah	2.60
Long Farm, Idaho	2.40
Gage Canal, Cal	2.24
Canal No. 2, Wyo	2.53
Vance Farm, Ariz	2.82
Biles Lateral, Colo	1.82
Middle Creek Ditch, Mont	2.10
Gothenburg Canal, Neb	2.47
Mean of all the above	2.37

An interesting comparison with the results in Table No. 1 is afforded by the mean of the duties on all the distributaries of the Ganges Canal for 1889–90, during the Khareef season, as reported in Buckley's "Irrigation Works in India." Here, the mean volume of water used in the irrigation of an acre of land was 121 970 cu. ft., equal to a depth of 2.8 ft. for the area irrigated.

Where the water was measured at the margin of the fields, there was a still higher duty than where measured at the heads of the laterals. Table No. 2 shows the duty obtained where all losses in distribution were eliminated, and nothing but the water actually spread over the fields was measured.

DIAGRAM SHOWING THE TIME OF IRRIGATION AND THE DEPTH OF WATER USED ON THE FARM OF OLEF CRONQUIST, LOGAN, UTAH.

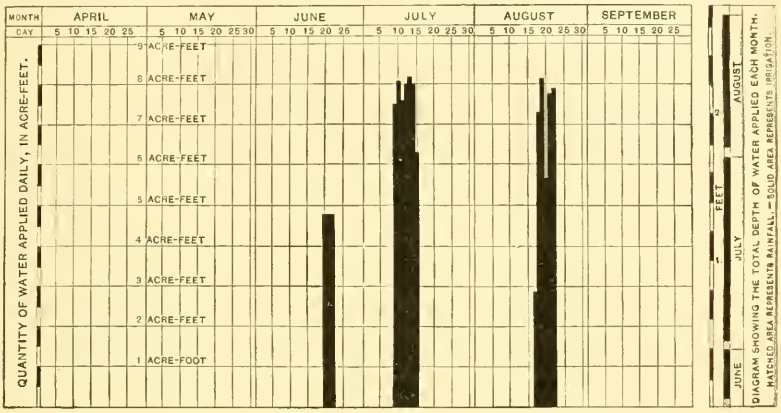


FIG. 7.

DIAGRAM SHOWING THE TIME OF IRRIGATION AND DEPTH OF WATER USED ON THE FARM OF A. F. LONG, NEAR NAMPA, IDAHO.

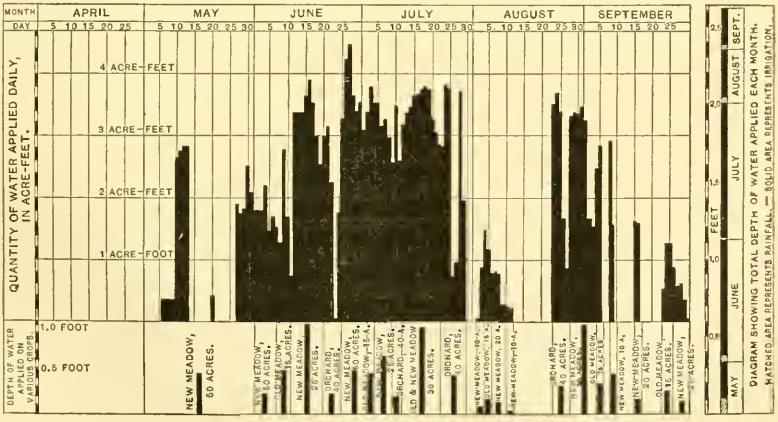


FIG. 8.

TABLE No. 2.—Quantity of Water Used, where Measurements Were Made at Margin of Field where Used.

Location.	Depth of water used, in feet.
J. Lateral, Wyo. (Oats)	1.55
J. Lateral, Wyo. (Corn)	0.70
Farm, Edgar Wilson, Idaho	1.48
Lowest Division, Gage Canal, Cal	1.78
Mean of measurements at Bozeman (Mont.), Experiment Station	1.20
Mean of all the above	1.34

The dates when water was applied, the length of the irrigation periods, the total acre-feet per day in use, the total depth of irrigation applied each month and the depth of rainfall for each month, are all shown graphically in Figs. 7, 8 and 9, for three of the stations named in Tables Nos. 1 and 2.

Losses by Seepage and Evaporation.

The duties given in the foregoing tables were obtained on laterals, or on canals where the losses in transit were not large, and on fields where the water was measured at the margin. They therefore represent, approximately, the volume utilized. In practice, however, the losses in canals, from percolation, leakage of flumes, evaporation, etc., are an important factor in fixing the average duty of water from a river or an extensive canal system. To determine this average duty, the volume should be measured at the headgate, and the number of acres it irrigates is the duty which canal managers have to consider in determining the area their works will irrigate. This duty is much lower than that obtained by measurements made on laterals or at the margins of the fields where used, the influence of the losses between the headgate and the heads of laterals being greater than has usually been supposed. Where canals cross gravel beds or gypsum deposits the results closely resemble trying to carry water in a sieve. Table No. 3 gives the depth of water used in irrigation, where the measurements were made at the canal headgates, and includes the loss from seepage and evaporation.

DIAGRAM SHOWING THE TIME OF IRRIGATION AND DEPTH OF WATER USED FROM THE GAGE CANAL IN DISTRICT NUMBER ONE, RIVERSIDE, CALIFORNIA.

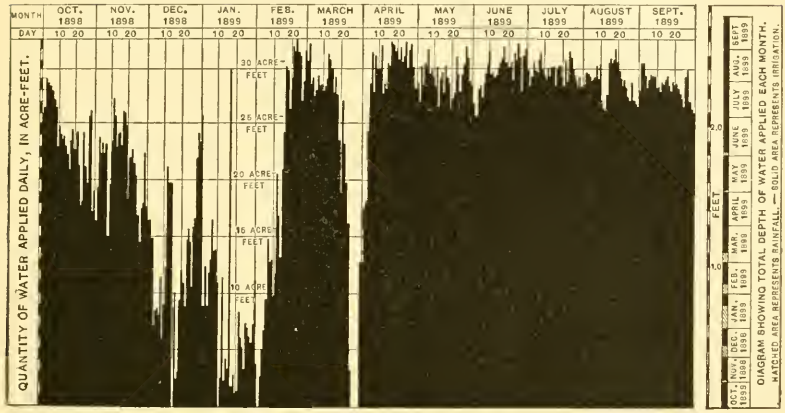


FIG. 9.

DIAGRAM SHOWING THE TIME OF IRRIGATION AND THE DEPTH OF WATER USED FROM THE BIG DITCH, SALT LAKE CITY, UTAH.

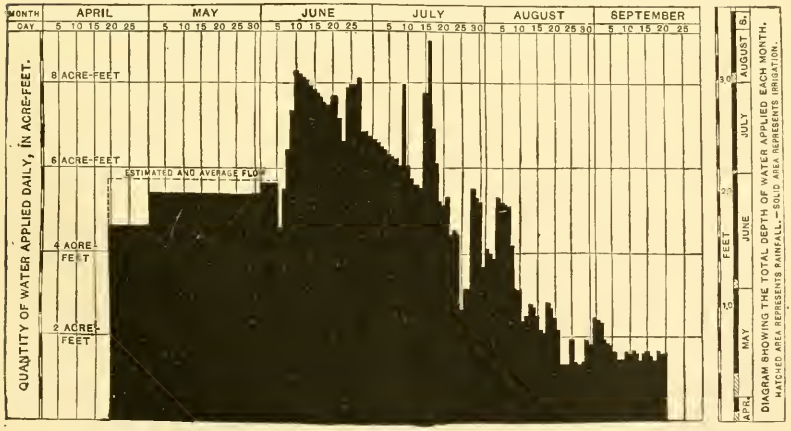


FIG. 10.

TABLE No. 3.—Depth of Water Used in Irrigation, when Losses in Main Canal are Included.

Name of Canal.	Depth of water used, in feet.
Pecos Canal, N. Mex	6.26
Mesa Canal, Ariz	3.81
Butler Ditch, Utah	6.24
Big Ditch, Utah	3.09
Upper Canal, Utah	6.30
Amity Canal, Colo	4.92
Rust Lateral, Idaho	5.06
Mean of all the above	5.10

For the Big Ditch, Salt Lake City, Utah, Fig. 10 shows when the water was applied and the total acre-feet applied per day, also the depth of irrigation water applied each month, and the rainfall for the same period.

A comparison of the duties in Table No. 3 with those obtained when the water was measured where used, will show that more than twice as much water was required where it was measured at the headgate as where measured at the place of use. In other words, the losses in the canals from seepage and evaporation amount to more than half the entire supply. This is in accord with many of the measurements made on irrigation canals in India. Among those recorded in Buckley's "Irrigation Works in India" is one which shows that the irrigation of wheat under the Jamda Canal in Bombay required 5.6 acre-feet of water for each acre irrigated, where the water was measured at the head of the canal, but where the water was measured at the place of use it required, in two experiments, only 2.1 and 1.4 acre-feet, respectively, to irrigate an acre, the loss in the canal being more than 50 per cent. On the Hathmati Canal, in the same country, the loss from seepage and evaporation was 50 per cent. The losses in transit in our canals are much heavier than is the rule on the older canals of India, and are doubtless more general than they will be in this country when the banks of canals are older and when they are operated with greater regard for economy.

The reports of the special agents and observers in charge of these

measurements show that the losses from seepage and evaporation, where measured in 1899, were as follows:

Pecos Canal, N. Mex., loss	52.3%
Mesa Canal, Ariz., estimated loss	25.0%
Middle Creek Ditch, Mont., loss in 4 miles	22.0%
With probable loss for entire ditch of	35.0%
Jordan and Salt Lake Canal, Utah, loss	45.0%
Gage Canal, Cal., loss	0.5%

The Gage Canal, being cemented, the losses from seepage are practically nothing. As compared to losses varying from 25 to 75%, shown in other canals, the loss of only half of 1% in this canal has great significance. The water turned into the head would have served to cover the land irrigated to a depth of 2.24 ft., while the mean depth for the water delivered to irrigators' laterals was 2.11 ft., a loss of only 0.13 acre-foot per acre irrigated. Canals can only be cemented on earth, as is done in California, in localities where frosts in winter are not severe.

Relation of Losses in Transit to the Amounts of Appropriations.

This loss from canals has given rise to a number of perplexing questions regarding appropriations. The states of Nebraska, Wyoming and Idaho have fixed a maximum limit on the amount of water per acre which an appropriator can acquire. In two of these states no one is permitted to appropriate more than 1 cu. ft. per second for each 70 acres irrigated. This limitation has given rise to the question as to whether the water so appropriated is to be measured at the margin of the irrigated field or at the head of the canal. Appropriators have claimed that if measured at the headgate the losses in the canal will be so great that the amount delivered will be inadequate for their needs; while water commissioners have insisted that it will be practically impossible to measure each appropriation at the head of the user's lateral. Several contests over this matter have already arisen, and sooner or later an authoritative decision will have to be reached. In one instance all the water entering the canal is delivered at a point 11 miles below. Measurements were made to determine the loss in this distance, and it was found to vary from 18 to 36 per cent. It was urged that this loss was so great that the amount turned into the canal should be increased enough to compensate therefor, but the State authorities refused to

recognize this claim, on the ground that such a concession would put an end to the improvement of ditches, since the greater the loss, the larger the appropriation which would be secured, and that its practical effect would be to place a premium on poor construction and wasteful operating of canals.

Seepage Waters Reappearing in Springs and Ditches.

Much of the water which escapes from canals finds its way to the surface below in the form of springs in what were originally dry ravines. Irrigators have filed on these springs, and secured thereby an ample water supply without having to pay the canal company which furnishes it anything for operating expenses or for the purchase of a water right. On the South Platte River alone there are over 400 of these filings on seepage waters. The report of the State Engineer of Colorado for 1898 shows that 5 000 acres in the Poudre Valley were irrigated with seepage water in that year. In a number of instances canal companies have sought to establish a title to the water of these springs and to collect for its transportation from their users; but the decisions of the courts in these cases have been conflicting, and no settled policy has as yet been established.

In some cases, where slopes are crossed by several canals, the lowest one frequently is benefited rather than injured by filtration, as it intercepts the water lost above. In one instance, where it is known that a large volume of seepage water is escaping from high-line canals, a ditch has been cut parallel with the river bank, but some distance away from it, to intercept this percolating supply. This has led to litigation to determine whether or not this is an interference with the rights of prior appropriators below on the main stream.

The percolating waters from canals and irrigated fields materially increase the water supply of western rivers. Measurements of this return or seepage water have shown that this reaches in many instances 30% of the original volume. In some cases it exceeds the original supply.

Influence of Fluctuations in Supply on the Duty Obtained.

A low duty does not of necessity indicate a wasteful or unskilful use of water. An illustration of this is found on streams which furnish more water than can be used during the flood season, but where the

period of plenty is followed by an equally assured period of drought. Irrigators have learned to provide against the latter by pouring on their land all the water it will hold while it can be had. By thus saturating the subsoil they store up a reserve supply which plants draw upon when the ditches fail. The report of one special agent deals with this practice, and a study of the flow of water in the Mesa Canal shows how marked is the scarcity during the hottest part of the year, and that at the time when the most water would be used if it could be had, less was actually used than at any other time in the season. That it is better to waste water on the land when there is a surplus, than to let it escape down the river and have crops burn later in the season, is beyond question, but the results of such irrigation are not nearly so satisfactory as they would be if this flood supply could be stored in reservoirs and be available for use when needed. Mr. Code's report shows clearly the necessity of reservoirs in localities like the Salt River Valley. They are needed to store the floods which now run to waste. They are needed to enable farmers to use water when crops demand moisture, and to relieve them from the alternating floods and drouths which dependence on the stream alone renders inevitable.

The Difference Between the Assumed Duty in Canal Contracts and the Duty Found by the Year's Measurements.

A comparison of the results of the year's measurements with the duty of water assumed in many important water-right contracts is interesting as showing their agreement or departure from actual practice. The quantity of water agreed to be furnished by a number of canal companies is as follows:

Arizona.

Arizona Water Company sells perpetual water rights, for $0.83\frac{1}{3}$ cu. ft. per second to be used on not to exceed 80 acres. This gives a duty of a little less than 100 acres per cubic foot per second, or a depth of 7.5 ft. on the land irrigated.

Consolidated Canal Company sells perpetual water rights for not to exceed $\frac{1}{3}$ miner's inch per acre. This gives a duty of 120 acres per cubic foot per second, or a depth of 6 ft.

Rio Verde Canal Company sells water by quantity, agreeing to furnish sufficient water to cover land to a depth of 2 ft. if it is called for, and more at the option of the company.

California.

Gage Canal allows 1 in. to 5 acres, or 1 cu. ft. per second to 250 acres. This water is not delivered in continuous flow, but in large streams for short periods, at the convenience of the consumer. This flow gives a depth of 2.89 ft.

Colorado.

Larimer County Ditch sells water rights for $\frac{1}{60}$ of the capacity of the ditch, without specifying how much land is to be irrigated.

New Loveland and Greeley Irrigation and Land Company sells one water right for each 80 acres, allowing a flow of 1.44 cu. ft. per second. The Colorado law compels companies to furnish water from April 1st to November 1st. For that length of season 1.44 cu. ft. per second would cover 80 acres to a depth of 5.3 ft.

Platte Valley Irrigation Company sells a water right for each 80 acres, allowing a flow of 1.44 cu. ft. per second, or a depth of 5.3 ft.

Fort Morgan Land and Canal Company sells a water right for each 80 acres, allowing a flow of not to exceed 1.44 cu. ft. per second for the irrigating season, or a depth of 5.3 ft.

Arkansas River Land, Reservoir and Canal Company sells a water right for each 80 acres, allowing a flow of not to exceed 1.44 cu. ft. per second, or a depth of 5.3 ft.

Dolores No. 2 Land and Canal Company sells water by the cubic foot per second, without limiting the consumer as to the area which he may irrigate.

Idaho.

Phyllis Canal contracts provide that the amount of water delivered shall not at any time exceed an amount equivalent to 1 cu. ft. per second for 50 acres, and that the total maximum quantity allowed shall not exceed 2 ft. in depth on the land irrigated; since 1899, will sell water by the cubic foot per second, with no limitations as to the area to be irrigated.

Boise and Nampa Canal, until 1899, delivered water at the rate of 1 miner's inch to the acre, but not to exceed 3 ft. in depth on the land irrigated. Since 1899, sells water by the cubic foot per second, with no limitations as to the area to be irrigated.

Montana.

Minnesota and Montana Land and Improvement Company sells water by the miner's inch without regard to the area to be irrigated. For the season of 1899 the farmers under this company's canal ordered, on an average, 1 miner's inch for 3.77 acres, showing a duty of about 150 acres per cubic foot per second.

Nebraska.

North Platte Irrigation and Land Company sells water rights for 1.44 cu. ft. per second, and prescribes land on which it shall be used.

Interstate Canal and Water Supply Company (Wyoming and Nebraska) agrees to furnish 1 in. per acre, or 1 cu. ft. per second for 50 acres. The legal season in Nebraska is 200 days—from April 15th to November 1st. One cu. ft. per second will cover 50 acres to a depth of 7.9 ft. in that time.

New Mexico.

Pecos Irrigation and Improvement Company water-right contracts provide for the delivery of 43 560 cu. ft. of water per acre, sufficient to cover the land to a depth of 1 ft., delivered at such times and in such quantities as may be necessary for the proper irrigation of crops.

Texas.

T. C. Purdy's water-right contracts call for the delivery of 43 560 cu. ft. per acre, to be delivered in not more than five irrigations per annum.

Washington.

Yakima Investment Company contracts to deliver not to exceed 1 cu. ft. per second per 160 acres, from April 1st to October 31st. This gives a depth of 2.65 ft.

Wyoming.

Wyoming Development Company sells water rights, giving the right to part of the total flow of canal.

Fetterman Canal Company sells a water right for each 8 acres, allowing a flow of $\frac{1}{10}$ cu. ft. per second. Wyoming canals do not ordinarily run more than 60 days. In that time this flow would give a depth of 1.46 ft.

Cody Canal shares represent water for 40 acres, and the quantity delivered is not to exceed $\frac{1}{2}$ cu. ft. per second. This gives a depth of 1.46 ft. in 60 days.

Need of Continuing the Investigation.

The investigations of the past year did not deal with all the factors which influence the duty of water, because time did not permit of their study. For these reasons, the necessity for storage reservoirs has been left for subsequent study and discussion. The subject is

of commanding importance, because it cannot be considered apart from the fundamental question of who is to own or control the stored water, and likewise the stream from which it is taken or along which it is impounded. Sooner than is generally realized, the public or private ownership of western rivers is destined to be one of the great social and industrial questions of this country. Their waters are worth more than the land, public or private, through which they flow, and the manner of their disposal will do more to shape western civilization and promote or retard western development than all other causes combined. The study of the duty of water is a study of the farmer's needs, and it is hoped that the presentation of these needs will tend to promote the creation of an irrigation system which will make the supplying of his necessities of first importance, and be a matter of just pride to the nation.

TABLE No. 4—Summary of Measurements of Duty of Water, 1899.

Station.	Period during which water was used.	Rainfall.	Evaporation.	General Duty.		Special Measurements of Duty.			Remarks.
				Depth of irrigation.	Depth of irrigation and rainfall.	Location.	Depth of irrigation.	Depth of irrigation and rainfall.	
		Feet.	Feet.	Feet.	Feet.		Feet.	Feet.	
Carlsbad, N. Mex.—									
Pecos Canal	Entire year	*0.31	4.55	6.26	6.57	Hagerman Farm	15.44	15.75	Measurements carried on from April to October.
Mesa, Ariz. (Mesa Canal)—									
1896	"	0.78	.....	5.91	6.69				
1897	"	1.04	.....	5.55	6.59				
1898	"	0.57	.....	5.01	5.58				
1899	"	0.39	.....	3.81	4.20				
Riverside, Cal.—									
Gage Canal	"	0.47	.....	2.24	2.71	District No. 1	2.32	2.79	
						District No. 2	2.23	2.70	
						District No. 3	1.78	2.25	
Salt Lake City, Utah—									
Butler Ditch	April-September	0.49	.....	6.24	6.73				
Brown & Sanford Ditch	"	0.49	.....	5.32	5.81				
Upper Canal	"	0.49	.....	6.30	6.79				
Green Ditch	"	0.49	.....	4.52	5.01				
Lower Canal	"	0.49	.....	2.83	3.32				
Big Ditch	"	0.49	.....	3.09	3.58				
Logan, Utah—									
Logan and Richmond Canal	June-September	0.27	3.27	3.59	3.86	Cronquist Farm	2.60	2.87	
Lamar, Colo.—									
Amity Canal	March-September	0.91	.....	4.92	5.83	‡ Biles Lateral	1.82	2.50	
Gothenburg, Neb.—									
Gothenburg Canal	June 7th-September 30th	1.26	.....	2.57	3.83	§ Daggett Farm	4.24	5.50	
Wheatland, Wyo.—									
Canal No. 2	June-August	0.37	1.26	2.53	2.90	J. Lateral	.....	.....	Evaporation at Laramie, Wyo.
						Oats	1.55	1.92	
						Corn	0.70	1.07	
Boise, Idaho—									
Boise and Nampa Canal	May-September	0.22	.....	.....	.....	Rust Lateral	5.06	5.28	
						A. F. Long Farm	2.40	2.62	Rainfall at Boise.
						Wilson Orchard	1.48	1.70	" "
Bozeman, Mont.—									
Middle Creek Ditch	June 16th-Sept. 16th	0.42	†1.74	2.10	2.52	Bozeman Exp. Farm			
						Clover	1.02	1.46	Rainfall 0.44 ft.
						Pease	1.10	1.51	" 0.41 "
						Grain	1.98	2.40	" 0.42 "
						Barley	0.98	1.39	" 0.41 "
						Oats	1.53	1.91	" 0.38 "
						"	1.34	1.70	" 0.36 "
						"	2.16	2.52	" 0.36 "
						"	1.28	1.72	" 0.44 "

*May 1st to November 11th, 1899. †Evaporation from July 6th to September 30th. ‡Water used under Biles Lateral, April-September. Rainfall at Holly, Colo., 0.68 ft. § Rainfall, April to September. Water used after harvest.

DISCUSSION.

Mr Merriman. MANSFIELD MERRIMAN, M. Am. Soc. C. E.—In this paper the word "duty" seems to have been used in two senses which are not exactly the same. It is said that the number of acres irrigated by 1 cu. ft. per second is the duty of water. That appears to be an excellent definition, and one that agrees with the use of the word duty in other branches of engineering; but when it is said that 1 acre of land requires so many acre-feet of water to irrigate it, this is not properly the duty of water, but rather a reciprocal of duty. The speaker has found that in presenting engineering matters to students it is extremely important to have the units very clearly and closely defined, and although this word duty is perhaps the least important of all the units, the speaker believes that it may deserve the remark he has made concerning it.

In this connection, the speaker was glad to note that in the publications of this Society on hydraulic matters the terms "second-foot" or "foot-second," as some people say, are not used. These terms lead to confusion, while "cubic feet per second," which should be always used, is clearly understood by everybody. There are many hydraulic engineers, particularly in the Government employ, who always say "second-feet" in designating the discharge of water, which is contrary to the principles of formation of compound units. In such a unit, as a foot-pound, or ton-mile, the meaning of the hyphen is that the two simple units are multiplied together; and in the term "second-feet," it might be natural to think that the number of seconds should be multiplied by the number of feet, which would be absurd. On the other hand the word "per" means division; for instance, "cubic feet per second," means the total number of cubic feet divided by the total number of seconds, and is therefore clearly understood. The term "acre-foot" is an excellent compound unit, formed according to correct principles.

Mr. Hering. RUDOLPH HERING, Vice-President, Am. Soc. C. E.—It seems strange that up to the present time the relationship of irrigation and drainage to agriculture has received so little attention. It is only quite recently that proper observations have been made as to the quantity of water which various crops require at different times. The agricultural experiment stations in the United States have begun to make some observations on that subject, and similar stations in Europe are doing the same. Yet it is hardly possible to get reliable information, except from a few isolated points and regarding a few plants. For instance, in this country, the agricultural station at Madison, Wis., is the only one known to the speaker at which the amount of water required

during the different months by a certain crop has been determined, and they have there experimented with scarcely a dozen crops. Mr. Hering.

The speaker believes that it is very important for the agricultural, irrigation or drainage engineer, to know just what amount of water the different crops need and when they need it, particularly in those sections of our country where there is scarcity of water. It is known that during different stages of growth plants need different quantities of water. For instance, at first, during the period of germination, nearly all plants need more than immediately after they have started to grow. Then, they again need a larger quantity, in fact, the most liberal irrigation, when passing through the flowering stage, and the least quantity during fruiting and maturity, when too much water becomes rather injurious.

As this subject is one which the engineer in this country has to meet occasionally, the speaker, when in Europe this summer, made some inquiries thereon, but found a remarkable dearth of experience there also; yet every station is at work trying to get results on the subject.

There is a correlative subject upon which we also have scarcely any data: Where there is water in the ground which is to serve the roots, the question sometimes arises, how far can the ground-water stand below the roots in order that these may still receive sufficient moisture? In our immediate neighborhood there are at this day suits threatened for several hundred thousand dollars which depend upon that very fact, as to the relation of the quantity of water needed by certain crops, the capillarity of the soil, and the level of the ground-water. This is a question which we, as engineers, will eventually have to face. As yet, there is but little information on the subject, and it is very gratifying indeed to see that certain Government officers at the agricultural stations, and engineers abroad, are now studying this matter which, in our country, in one way and another, represents a great deal of money to agriculturists.

A. D. Foote, M. Am. Soc. C. E. (by letter).—Professor Merriman's remarks on the erroneous use of the term "second-foot," or "foot-second," are most heartily concurred in by the writer. It is hoped that the members of the United States Geological Survey may read it and take heed. They gave us the excellent term "acre-foot," Major Powell first using it; they also gave us the term "Nilometer" (Major Dutton), which is perhaps the most misleading name ever applied to a bit of clockwork. The Nilometer is a pillar on the banks of the Nile which has been watched by the Egyptians since beyond the time of any human record, telling them each year of disaster, plenty or famine as the water rose to the marks upon its sides. Mr. Foote

There can be no other Nilometer, and to apply its name to the weir register standing on the bank of the "Old Woman's Fork," in

Mr. Foote. Wyoming, is certainly something more than an anachronism. The "Old Woman's Forkometer" would be an analogous term, perhaps, and much more appropriate.

A matter worth recording in connection with Mr. Mead's weir registers, rather than as a comparison or criticism, is the following description of a register designed by Mr. G. F. Sherman, Assistant Superintendent of the North Star Mines Company. The main differences from ordinary weir registers are that the clockwork regulates the movement of the paper by holding it back instead of carrying it, and the rapid motion of the paper gives opportunity for recording very frequent variations; also, the roller marks lines on the paper so as to give an accurate and continuous scale of heights in perfect agreement with the pencil.

The Empire Mine, in Grass Valley, obtains its power chiefly from water delivered at the mine, by the North Star Mines Company, under a head of about 400 ft., which is applied to several impulse wheels, each running its own machine. In the routine work of the mine the use of water is extremely variable, making it necessary to have some indisputable record of its fluctuations in order to determine the actual flow.

The weir register here described, was designed for this purpose early in 1899, and made in the machine shop at the North Star Mine. It consists of two brass rolls carrying a strip of paper, wound from one on to the other, and which passes through a pair of governing rolls between them. It is driven by a weighted cord, and its speed is regulated by a clock movement geared to one of the governing rolls. The machine is placed over a water chamber connected by a pipe to the weir box. A rod attached to a copper float rises and falls with the variations in height of the water on the weir. It is guided at the top by an easy-fitting slotted bracket, and at the bottom by a rod which plays in a pipe passing through the float.

The pencil it carries is pressed against the paper by a light spring contained in the bracket. A fifth roll, not absolutely essential, supports the paper against the pencil, and keeps it smooth and in a fixed position, which would not be the case if it worked against one of the other rolls gradually increasing or diminishing in diameter. The pencil marks the paper before it passes over the governing rolls in order to avoid catching in the grooves in the paper made by them. The float is large enough to require a weight of about 2 lbs. to sink it $\frac{1}{8}$ in. below its proper depth of flotation.

The other details of construction relate chiefly to the difficulties of constructing a true light-running machine with the facilities at hand. In order to allow for a slight eccentricity in the governing rolls, as well as the varying thickness of the paper used, one of the rolls revolves in sliding boxes which are pressed toward the other by

springs with adjustable tension. No clock, with power enough to make any impression upon the speed of the paper, even when most of the work was done by the weight, could be obtained, but it was found that a clock could stand a heavy drag, tending to accelerate it, and still run regularly; therefore, enough weight is put on the cord to run it too fast at all times, and the clock holds it back to a uniform rate. The paper moves 30.5 ins. per day, exposing at any time the record for the previous 12 hours. The record is removed once or twice a month, but the supply roll carries sufficient paper to run for 6 months. Mr. Foote.

The weir is 3 ft. long, rectangular, with suppressed end contractions, and is not expected to measure a flow of more than 10 cu. ft. per second.

The daily average height is determined by measuring the area above a known height and below the record line, with a planimeter, and dividing by the length. To assist in getting the areas, and also in estimating the flow at any time, ridges are turned on one of the governing rolls, which mark the paper in lines 0.05 ft. apart, the lowest line corresponding to zero on the weir. This method of obtaining average heights from areas is, of course, not quite correct, but is a rapid way of getting a result accurate enough for practical purposes.

Elwood Mead, M. Am. Soc. C. E. (by letter).—The cubic foot per second is not a satisfactory unit in studies or discussions of the duty of water. On the contrary, it is nearly always misleading. It assumes that water is used continuously and at a uniform rate of flow, while, in fact, it seldom is. In the studies made at the Experiment Stations of the United States Department of Agriculture last year there were some farms on which the water was only used for seven days; on others for more than three hundred. In some cases it would be used only a few hours at a time, with alternating periods of non-use or partial use. Hence, the discussion of the duty of a cubic foot per second of water would have been wholly theoretical. On the other hand, a discussion of the duty of an acre-foot involves no misleading or purely arbitrary assumptions. It is just as definite a unit as the cubic foot per second, and has the advantage of eliminating the element of time and rate of delivery in the comparison of results. Hence, the acre-foot is used in describing these investigations. Sometimes it would seem to be clearer to speak of the depth to which the water used would cover the land. Where this is done it is not the duty of the acre-foot, but the reciprocal of the duty, which is referred to, as Mr. Merriman has pointed out, and that explanation is made in the first report of the investigations, but was overlooked in the paper. So understood, no misconception can arise. Mr. Mead.

The relationship of irrigation and drainage to agriculture has, as

Mr. Mead. Mr. Hering points out, been strangely overlooked in this country. Since taking charge of these investigations for the Office of Experiment Stations the writer has been doing all he could to promote the study of agricultural engineering in the Agricultural Colleges of the arid region, but finds it difficult, at times, to make the authorities understand that this subject comes properly within the province of these schools, and that they are different from schools of hydraulic engineering. The moisture studies, referred to by Mr. Hering, have already been inaugurated in California, and, from the facts gathered, it is believed that they are destined to greatly benefit the agriculture of that state. In the San Joaquin and Sacramento Valleys, where both temperature and evaporation are excessive, certain crops are successfully grown with a rainfall which varies from 6 to 16 ins. How this is accomplished has long been an enigma to the writer, but one explanation seems to be that much of the water needed by plants is pumped up by the roots from several feet below the surface, where the soil is not moistened by rains but by the water which percolates from streams.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

No. 883.

A PROPOSED METHOD FOR THE PRESERVATION OF TIMBER.

By F. A. KUMMER, Jun. Am. Soc. C. E.

PRESENTED JUNE 6TH, 1900.

WITH DISCUSSION.

In presenting these notes upon a proposed method for the treatment of timber to preserve it from decay, the writer fully realizes that they may seem to add but little to the mass of information already collected upon that important subject; but the experiments which are herein described and the direction in which they lead, indicate a line of improvement in the preservation of timber which will prove of interest.

So many figures have already been presented, proving the great need of a successful method of timber preservation and the immense value of such a process—could one be secured, giving practical results at a reasonable cost—that the means by which this result may be obtained may be at once considered without stopping to prove its necessity.

The experiments herein described, and the lines upon which the proposed process has been developed, were made, particularly with a

view of improving what is at present known as the creosoting process, it being believed that this process would command a much more extensive use if certain mechanical difficulties, now met with in connection with its application to timber, and particularly to railroad ties, could be removed. The improvement of this process was also attempted because of its acknowledged merit when properly applied. As at present applied, it possesses certain features which preclude its extensive use in the treatment of railroad ties. If, by a simplification or improvement of the process as now used, the objections hitherto raised to it may be overcome without sacrificing any of the advantages which it possesses, and if, further, this can be accomplished at no greater cost, the resulting process would commend itself to engineers generally.

The often-quoted report* of the Committee of this Society, presented in 1885, said in part:

"If the exposure is to be that of a railroad tie, creosoting is doubtless the most perfect process to use; but in view of the expense, it may be preferable to use a cheaper process, dependent somewhat upon the location."

It is probable that at the present time a greater expense for treatment would be warranted by the cost of timber, as well as by the general conditions of roadbed maintenance, particularly upon eastern roads. Without here entering upon a discussion of the expense for preservation which would be justified under given conditions, it would perhaps be better to discuss first the proposed process from a technical standpoint, and, after arriving at its cost, determine to what extent a process of a given cost per tie could be used.

The creosoting process as at present applied in one of the largest, if not the largest, creosoting works in the world, is described in part as follows in the printed publications of that company:

"The Bethel process, the first to use dead oil of coal-tar, consisted of drying the timber naturally, then applying cold vacuum. This takes too long a time, as large timbers, to dry in this manner, take from 3 to 6 months. Consumers of creosoted timber in this country cannot wait for timber to dry naturally, and so steam heat was introduced * * * to hasten the process. By the use of steam, assisted by superheating, the process is reduced to a small fraction of time. By our process timber can be used fresh from the stump or water, and in a few hours the sap and moisture extracted by means of our

* *Transactions*, Am. Soc. C. E., Vol. xiv, page 290.

immense steam boilers and coils, also by our powerful vacuum pumps, and be prepared for impregnating with oil. * * *

"The timber is first loaded on cars and run into the cylinders which are then hermetically sealed with immense iron heads. Steam is then admitted into the cylinder and surrounding the timber, superheated steam is also introduced into the cylinders by means of large coils so that it does not come in contact with the timber, and the heat is maintained until the timber is heated all through at a low temperature so as not to injure the woody fibers. The cylinder is then freed of all vapors, and the vacuum pumps are put to work to exhaust all the sap and moisture, which is then in the shape of vapor, from the cylinder. Heat is maintained in the coils to prevent the vapor from condensing and thereby remaining in the timber. As the vacuum pumps are constantly removing the hot vapor from the timber it is absolutely necessary to keep the heat above the condensing point. To do this requires practical experience and means of knowing what such heat is, and, as said before, these two parts of the process are the most important, and, if properly done, the oil will be readily forced into the timber. After this has been done the oil is admitted into the cylinders while they are under vacuum, and when all the air has been withdrawn they are subjected to pressure until the requisite amount (which is determined by correct gauges and thermometers) has been forced into the timber, which, if the timber has been properly prepared, is only a small part of the process; but if this has not been well done, the oil cannot be put into the timber. The cells of healthy timber are full of different substances, which, when subjected to heat, can be changed into vapor, and unless the vapor has been completely removed you cannot force the oil into the timber, no matter how long the pressure is applied. It is only by practical knowledge and delicate instruments that we determine when the heat has reached the center of the timber, and the vapor there formed has been removed.

"There will be no decay in any part of the timber that has been permeated with the oil, but to have all parts saturated is expensive and useless; for, after the timber has been thoroughly treated by the heat and vacuum process it will last a long time without any oil, and if the crevices and pores are sealed up with the oil to a sufficient depth the timber is as good as if the whole part had been thoroughly permeated with the oil. The quantity of oil to be used should be determined by the use to which the timber is to be subjected."

Before making any comments upon the process just described, the writer will give the reasons for the decay of timber, and state exactly what steps should be taken to prevent this decay. Having done this, he will analyze the dead oil of coal-tar process as at present used, and see whether these necessary steps are carried out properly.

There are two causes for the decay of timber. *First*, through the fermentation of its contained sap, such fermentation being caused by the germs contained or developed in the sap. *Second*, through the introduction of germs of decay from without, through the action of air or water. As Mr. Curtis says:*

"Ties perish by mechanical destruction of the fibers under the rail; splitting by seasoning and cutting out of fiber by respiking; and by natural decay. In its normal condition, to resist these three influences, a timber is required which shall have sufficient hardness to resist the mechanical injury, a high degree of coherence between fibers, and a sap of such a chemical composition as will offer the least encouragement to the development of the fungoid life which destroys the fiber."

To treat a tie satisfactorily, it must first be sterilized throughout; removing at the same time as much of the contained moisture as is possible without injury to the fiber of the wood. Fermentation cannot then take place from within. It must then be treated in such a manner that germs of decay cannot enter from without, or, if they do enter, that the conditions for their development are highly unfavorable. It must further be provided that the antiseptic injected will not be dissipated through any agency, and, if the timber is to be subjected to the crushing effect of heavy railroad traffic, it must in some way be provided that the timber so protected will be able to resist the weights which will pass over it, without excessive crushing.

If a tie be subjected throughout to a sterilizing temperature of 212°, the agencies of fermentation or decay existing in the sap will be killed, and, if prevented from again entering the timber from without, decay cannot take place. This leads to the first important conclusion; namely, that complete sterilization throughout is a necessity in the treatment of timber. Without such sterilization no form of treatment can be satisfactory unless the timber be completely impregnated throughout its entire mass with an antiseptic material, which is rarely if ever accomplished with large timber. Such sterilization means fully 212° at the heart of the stick. Wood is a poor conductor of heat, and, to secure 212° or more at the heart, two ways are open: *First*, long exposure to a temperature of say 215 to 225° Fahr., which is expensive because of the time consumed; *second*, exposure to a temperature of 290 to 300° Fahr., which, under ordinary conditions, will

* *Transactions*, Am. Soc. C. E., Vol. xlii, p. 288.

injure the fiber of the wood, or at least induce extensive splitting or cracking. The necessity for sterilizing is not brought out in the description of the creosoting process previously given, and it seems to be a point, the vital nature of which is not generally recognized. Experiments made upon yellow pine ties, 6 ins. x 8 ins. x 8½ ft., having self-registering thermometers imbedded at the heart by means of an auger hole, 24 ins. deep, bored from the end, showed that with a temperature of 230° Fahr. in the cylinder, and at the surface of the tie, 178° Fahr. was secured at the center in two hours, while with 290° Fahr. at the outside, 249° Fahr. was secured in two hours. Starting with sterilization as an absolute necessity, it is evident that, if such a temperature as 290° can be used without injury to the timber, complete sterilization will be effected, and in a reasonably short time. It would be well to state here that, with a few exceptions, the experiments were made on long-leaf yellow pine of the best merchantable grade, the exceptions being several experiments upon white oak timber, which showed that it is not capable of treatment by such a process, owing to its great tendency to split and crack under high temperatures.

The method adopted to render the use of such high temperatures possible in the treatment of yellow pine is the application of pressure simultaneously with the application of heat. When the oors are closed the heat is raised to 215° Fahr. without pressure, taking one hour to accomplish this, and kept for one hour at 215° without pressure. This is for the purpose of getting rid of the moisture. Then the heat is increased, pressure is applied, and both are raised gradually, to avoid injury to the fiber, for two hours, until the heat has reached about 285° and the pressure about 90 lbs., and both are held there for one hour. The heat is then shut off, and the tanks are allowed to cool gradually for one hour. At the end of this time the heat is reduced to 250° and the pressure to about 40 lbs. The pressure is then blown off and the heat still further reduced. Vacuum is then applied until about 26 ins. is reached, and while under vacuum the mixture is run into the cylinder at a temperature of 175 to 200° and hydraulic pressure applied reaching 200 lbs. per square inch, and kept at this point until the desired amount of the mixture is absorbed. The liquid is then run off, and the wood is placed in another cylinder, and milk of lime, at a temperature of about 150°, is run in, and hydraulic

pressure of about 200 lbs. is applied for from one-half to one hour. The following is a condensed table of the treatment:

1 hour to reach	215°	Fahr.
1 hour under	215	"
2 hours (heat and pressure applied gradually) to reach	285	"
1 hour (90 lbs. pressure) at	285	"

1 hour cooling.

1 hour vacuum (26 ins.).

½ hour filling cylinders with mixture.

3 hours pressure (average 8 to 10 lbs. absorption).

½ hour emptying tanks.

1 hour under lime and pressure treatment.

The time will be increased or decreased, in keeping with the amount of absorption required.

After the preliminary treatment, the temperature of the interior of the cylinder is raised to 285 or 290° Fahr. If this temperature were maintained at atmospheric pressure, not only would the sudden and rapid evaporation of the remaining moisture in the stick induce splitting and cracking, but the volatile oils in the wood would be driven off, and this is not desirable, inasmuch as they play a not unimportant part in the preservation of the timber. Air pressure is therefore maintained in the closed cylinder at about 80 to 90 lbs. per square inch, so that while the tie is sterilized by heat, the natural oils are not driven off, and splitting and cracking does not result, while the heat is at the maximum.

One of the benefits of the vacuum is to exhaust the air which was forced in while the wood was under pressure, and which, if not removed, would have to find its way out subsequently either through the mixture or by creating cracks.

Mr. J. W. Putnam, in a letter to Octave Chanute, M. Am. Soc. C. E., Chairman of the Committee on the Preservation of Timber, in 1885, wrote as follows:*

"The most carefully conducted experiments indicate that there is no decay without fermentation, and no fermentation without germs. If a piece of timber be cut green and thoroughly coated with paint, it will soon be destroyed by what is called dry rot. If a similar piece be heated through to 225° Fahr., and a sufficient amount of oil be forced

* *Transactions*, Am. Soc. C. E., Vol. xiv, 1885, p. 337.

in to form an impervious coating, no decay will take place until that coating is broken."

This not only evidences the necessity of complete and thorough sterilization, but brings us to the next step in the process, the impregnation of the timber with the antiseptic material. Having thoroughly sterilized the tie, the natural continuance of the process is, of course, the application of the vacuum, to remove from the pores of the timber the air or vapor which they contain and admit the preservative fluid under this vacuum.

The preservative fluid used is not creosote, or dead oil of tar alone, but consists of dead oil of coal-tar, 38%; formaldehyde, 2%; and melted resin, 60%, by weight. The specific gravity of the resulting mixture at 300° Fahr. is 1.068. The resin is used to render the mixture absolutely waterproof, the formaldehyde to strengthen the antiseptic nature of the compound, necessarily somewhat reduced by the reduction in percentage of the dead oil itself. Upon this point the following extracts* from a letter by Mr. Edward R. Andrews to Mr. Chanute will be of interest:

"Creosote oil is a distillate of coal tar—a residual product in the manufacture of coal gas. Chemists have procured from coal tar a vast number of sub-products and combinations of great usefulness in dyeing, etc. The three principal coarse products of coal tar are the light oils, the heavy oils and pitch, all the results of distillation.

"The light oils (lighter than water) evolve in the distillation at a temperature of 360 to 480° Fahr. From these all the aniline colors are made. They are expensive and have no value whatever in wood preservation. The heavy oils (heavier than water) are distilled at a temperature of from 480 to 760° Fahr. These are the so-called creosote oils, and contain all the constituents of the coal tar useful in wood preservation. After the creosote comes the pitch. Creosote contains about 5% of tar acids, *i. e.*, carbolic, cresylic and other acids, but the bulk is made up of semi-solid oils and naphthaline.

"Wood preservation by the metallic salt processes is solely chemical. Earlier, it was claimed that the zinc chloride, etc., formed insoluble chemical combinations with the albumen contained in the sap wood. Now, it is generally allowed that no such combinations are formed, but that the value of metallic salts as antiseptics depends upon their continual presence in the woods, and as they are readily dissolved out of the wood their effect is only temporary. The life of wood is prolonged by their use, when skillfully applied, yet in moist places they quickly lose their efficacy.

* *Transactions*, Am. Soc. C. E., Vol. xiv, Appendix No. 14, p. 341.

"The creosoting process is both chemical and mechanical. Besides the carbolic and other acids, it contains many other well-recognized antiseptic constituents; but it is probable that the very long life of timber secured by thorough creosoting is due far more to the fact that the pores of the wood are filled up with the thick, gummy, insoluble oils and naphthaline, and thus keep out air and water, which contain the germs of decay. That such is the case was conclusively shown by M. Roltier, a Belgian chemist, and later, in 1866, by M. Charles Coisne, Chief of Section of the State Railways of Belgium, and Superintendent of the Creosoting Works.

"By the latter, two series of experiments were tried, during a period of five or six years, in burying in a compost heap made of decaying wood, manure, etc., shavings impregnated with creosotes containing different percentages of carbolic acid. The results showed that shavings saturated with carbolic acid alone were entirely decayed, and those saturated with the distillates at the highest temperatures which contained no carbolic acid whatever were perfectly sound.

"Experience with the metallic salts and the results of above experiments indicate that to preserve timber something more is required than an antiseptic for the purpose of coagulating the albumen. The very small percentage of albumen contained in the sap wood probably ferments readily and may originate decay; but the agencies of fermentation introduced into exposed timber by the air and water absorbed by the wood are vastly more dangerous than the seeds of decay contained originally in the wood itself.

"During the past hundred years almost every imaginable substance has been proposed as a preservative of wood, yet it may be that inventors are still at work, if so, their attention would be best directed to such methods or materials as would close the pores of wood to air and water."

This would indicate that creosote is more valuable because of its function in filling the pores of the wood and excluding air or moisture containing germs of decay than because of its direct antiseptic action. To a considerable extent, this is doubtless true, but the writer thinks that the ideal preservative should be both antiseptic and waterproof, and should possess the latter characteristic in the highest obtainable degree. To secure such a material was the chief object of the experiments which resulted in the process herein described, and the compound finally fixed upon consists of approximately 38% by weight of dead oil of coal-tar, 60% of melted resin and 2% of formaldehyde. The idea of introducing the resin was to obtain a substance in the highest degree waterproof, and there are few substances known more highly waterproof than pine resin.

This mixture is perfectly fluid at 200° Fahr., and, by some peculiar action not readily explainable, seems to be more readily forced into the wood than the creosote oil alone. There are a number of very important advantages gained by the use of the resin in connection with the dead oil. Timber treated by the ordinary creosoting process is more or less spongy and soft, and while the dead oil is not generally regarded as soluble, it is still subject to dissipation by the action of water, as is well known in the case of treated piles subject to the washing action of salt water. Experiments made by immersing in water timber untreated and treated by the process just described, showed results as follows:

One piece creo-resin timber, 6 ins. x 8 ins. x 8 ft., immersed in water 27 days, absorption $1\frac{3}{4}$ lbs. One piece, same stick as above, untreated, absorption $11\frac{1}{4}$ lbs. The increase in weight of the first stick was largely due to water clinging to the surface.

Timber treated with dead oil in conjunction with resin is practically absolutely waterproof and very much harder than the untreated timber, instead of being softer and more spongy, as is the case with creosoting proper. The introduction of bacteria from without should be prevented by a process as highly waterproof as it is possible to make it, and this process should use a material which will not only have these waterproof qualities, but will be so antiseptic in nature that the possibility of the existence or development of decay germs, if such germs by any means gain an entrance into the timber, should be reduced to a minimum. In fact, the conditions should be such as to absolutely preclude any possibility of their life or development. To secure these results, the combination of resin, creosote oil and formaldehyde is especially well adapted. The material weighs about 8.9 lbs. per gallon.

TABLE No. 1.

No.		Maximum Heat.	Pressure, in Lbs.	Pounds per cubic foot.
13 C.	Green.	285°	90	3.33
13 C.	Vulcanized.	285°	90	6.5
16	Green.	275°	80	7.14
16	Vulcanized.	275°	80	10.66
17	Green.	285°	85	3.43
17	Vulcanized.	285°	85	6.18
18	Green.	270°	80	6.75
18	Vulcanized.	270°	80	9.45
22	Green.	275°	90	5.22
22	Vulcanized.	275°	90	7.9

Experiments were made upon pine ties cut in two, one half vulcanized, the other half not vulcanized. The results are given in Table No. 1, and show the difference in absorption.

These specimens were treated with various degrees of heat and pressure, with and without vacuum, and in all cases they showed an increased absorption in the vulcanized over the unvulcanized timber.

The treatment of "green" lumber herein mentioned consists of simply putting the lumber in the cylinders and raising the heat to about the temperature of the liquid—say 200°—and then running the liquid in and keeping it there from 3 to 4 hours. Of course, this does not sterilize it or give as good absorption.

Other experiments were made upon ties, 6 ins. x 8 ins. x 8 ft., of long-leaf yellow pine, cut into three pieces, two of which were treated and the third left untreated. The results are shown in Table No. 2, the treated samples, 1 and 1-A, 2 and 2-A, etc., being the two pieces from the same stick.

TABLE No. 2.

	Absorption per cubic foot, in pounds.				Absorption per cubic foot, in pounds.		
	Above 10 lbs.	Bet. 8 and 10 lbs.	Below 8 lbs.		Above 10 lbs.	Bet. 8 and 10 lbs.	Below 8 lbs.
1	10.68			14	13.5		
1-A	10.12			14-A	14.62		
2	10.12			15	15.18		
2-A	10.28			15-A	15.74		
3		9		16	16.31		
3-A		9		16-A	13.5		
4			7.87	17		9.56	
4-A			7.87	17-A		9	
5			7.3	18		8.43	
5-A			7.3	18-A		8.43	
6			5.62	19	15.18		
6-A			5.62	19-A	15.18		
7	11.18			20			7.87
7-A	13.5			20-A		9	
8	12.96			22	13.5		
8-A	14.62			22-A	14.62		
9			7.87	23	16.31		
9-A			6.75	23-A	16.87		
10			6.75	24	12.92		
10-A			6.75	24-A	11.81		
11		9.56		25		9	
11-A			7.87	25-A		9.56	
12	12.93			26	13.5		
12-A	13.5			26-A	16.87		
13	12.37						
13-A	11.81						

As regards the spongy condition of the wood, after treatment by the creosoting process, an inspection of timber treated by the proposed process readily demonstrates that the timber is not in any way softened. The surface is subjected to treatment by the application of milk of lime after the creosote-resin has been injected. This tends to solidify the mixture of resin and creosote oil. Solidification to a point of brittleness is not effected, but solidification is effected to such a degree that the resulting surface of the wood is materially harder than that of untreated timber, instead of being softer, as is the case with creosoted timber. The writer has made tentative experiments to prove this hardening effect by striking treated and untreated sections of the same tie with a hammer. The greater density and hardness of the surface of the creosote-resin tie (about 50% increase in density near the surface) being readily apparent from the way in which it resists crushing under the hammer, the force of the blow causing the hammer to rebound from its surface instead of sinking in and crushing the fiber, as was the case with the untreated portion of the same tie.

Obviously, if it were commercially practicable to impregnate a railroad tie, of, say, long-leaf yellow pine, throughout its mass with creosote oil, no decay would take place in the timber so long as the oil remained in it. But such a treatment is impracticable, both because of its cost, and of the mechanical difficulties attending it. We therefore are obliged to fall back upon the plan outlined by Mr. Putnam in the letter previously quoted, namely, to secure complete sterilization, and to then inject sufficient of the preservative to effectually protect the remainder of the stick. This may be done by the injection of 8 to 10 lbs. per cubic foot in a standard-sized tie; 8 lbs., giving a penetration averaging at the center of the tie about $\frac{3}{4}$ to $\frac{7}{8}$ in., and 10 lbs., $\frac{7}{8}$ in. to $1\frac{1}{4}$ ins. at the center, with from 6 to 8 ins. inward from the ends of the stick in each case.

A tie treated by the above process, with say, 10 lbs. of preservative per cubic foot, possesses these advantages: It is thoroughly sterilized—a result not generally secured by the creosoting process—the outer fibers of the wood are filled with a material more highly antiseptic than dead oil of coal-tar, practically waterproof, and, when hard, considerably better able to resist crushing or cutting than is either the untreated or the creosoted timber. In fact, the spongy nature of creosoted timber, especially as the timber usually chosen for creosot-

ing is naturally soft, has been one reason why creosoted timber has not been more extensively used under heavy traffic. The experience of the Lehigh Valley Railroad was along this line. A tie treated by the process in question also possesses much higher spike-holding qualities—considerably higher than the untreated stick.

The writer has confined these remarks particularly to the subject of railroad ties; but it should not be overlooked that a large part of the business done by the creosoting works at present is upon piles and dock timbers; and the failure of such timbers when attacked by the teredo is invariable due to the washing out of the creosote oil by the action of the salt water. It would therefore be readily practicable to creosote piles, giving them either at the same time, or subsequently, a resin treatment to absolutely retain the creosote in the wood, and therefore the chief objection to creosoted piling would be at once removed.

The question of wooden paving blocks is also one having, in the writer's opinion, great possibilities. Wooden paving blocks, creosoted, are in extensive use in London and continental cities, and have given very favorable results in this country wherever properly used. If we add to the preservative effect obtained by the creosote oil the additional and most important benefit of so hardening the fibers of the wood that its resistance to wear is materially increased, we have realized a most material gain in the treatment of such blocks. In fact, if a treated pine paving block, protected against decay by dead oil of coal-tar, impervious to moisture and having considerably greater density and resistance to abrasion than either the creosoted or untreated timber, can be laid at a cost not greater than that at present charged for asphalt, it is the writer's opinion that such a pavement will possess all the advantages which asphalt possesses, and will be at the same time both noiseless and not slippery; thus removing two of the gravest objections to asphalt pavement. The writer thinks that such a pavement would have a longer serviceable life than asphalt—at least such as is generally laid—and on streets of a certain class be exceedingly desirable.

To revert once more to the question of tie preservation: With creosoting, as well as with the proposed improved process, the amount of impregnation per cubic foot is regulated by the specifications under which the work is done. For railroad ties the general practice is to use

about 10 lbs. Whether pure creosote or creosote and resin are used, this specification gives a depth of penetration at the center of the tie averaging nearly 1 in., and at the ends of the tie a complete penetration from the end of the tie of from 8 to 10 ins. Both with creosoting and with this process, the depth of penetration depends not only upon the amount of material injected per cubic foot, but also upon the nature of the timber and upon the individual qualities of different sticks of the same kind of timber. The writer has suggested as a possible disadvantage to such processes—whether creosoting or improved creosoting—the likelihood of the splitting of the tie taking place after treatment, thus allowing the introduction of germs into the heart of the stick. A careful examination and sawing of some fifteen to twenty ties, treated for periods extending up to 6 months, has shown no instance in which new cracks had opened in the timber and extended beyond the treated portion of the tie. Should this be the case after several years' service, the writer believes it to be true that the vulcanized heart of the tie treated by the improved process would better resist the action of decay than the untreated and unsterilized heart of the creosoted timber.

Further, it seems probable that water containing germs would to some extent get rid of such germs in passing through that portion of the timber which is antiseptic in its action. The results obtained with creosoted timber up to the present time do not indicate that such cracking or splitting, and consequent decay, are among the serious defects of the creosoting process. It seems reasonable to conclude that a tie which has been subjected to the continued high sterilizing temperature has developed splits in those weak portions where splits would otherwise have taken place under natural conditions, and these splits or cracks, developed in the earlier steps of the process, are filled with the antiseptic and waterproof material during the later stages, thereby excluding permanently both air and water.

DISCUSSION.

Mr. Prout. HENRY G. PROUT, M. Am. Soc. C. E.—This paper presents a proposition which is attractive, novel, and, so far as the speaker is aware, entirely original. Of course, the proposition to prevent the decay of wood by subjecting it to a sterilizing temperature is by no means new. About twelve years ago the process known as vulcanizing was exploited somewhat in New York City on precisely the theory advanced by Mr. Kummer, viz., that subjecting the wood to a high temperature would kill the germs, and that this temperature would solidify the albumen and other constituents of the sap and thus fill the pores of the wood with a solid, impervious, waterproof substance. The wood was treated under a high pressure also, to prevent checking. There should be by this time sufficient experience to show whether or not the expectations have been realized.

A large amount of vulcanized timber was put into use by the Manhattan Elevated Railway, in the tracks, station platforms and possibly elsewhere, ten years ago, or possibly longer. A considerable quantity was also put in service on the New York and Brooklyn Bridge by C. C. Martin, M. Am. Soc. C. E., and a large quantity was shipped to the Northwestern Elevated Railway of Chicago. From all these places we should now be able to get some information as to whether or not the vulcanizing process has any merit.

Obviously, however, the author thinks that vulcanizing is not thoroughly efficient. He must be quite familiar with its results, and yet he proposes to go further and supplement it by the addition of the antiseptic material and also the waterproofing resin. The speaker has always been skeptical as to the results of vulcanizing, because it has seemed inevitable that the transverse strength of the timber would be injured by heating it to such a high temperature.

In the early stages of vulcanizing, R. H. Thurston, M. Am. Soc. C. E., who was then at Stevens' Institute, made some tests of vulcanized sticks and reported that the transverse strength of the timber did not deteriorate under the process. The speaker has heard nothing further as to those results since Professor Thurston's tests.

Were it not for Mr. Kummer's statement, the speaker would also feel skeptical as to whether or not such a quantity of resin can be forced into the timber.

It is obvious that the high temperature treatment will develop checks in the timber—checks which are otherwise there, or would otherwise occur—and that those checks will be filled with creosote and resin which will, undoubtedly, add to the life of the timber.

The most important question is whether or not timber is actually injured by subjecting it to a temperature of about 300°, under pressure.

J. H. Deghuée, Esq.—Vulcanized wood, with which the speaker has had some experience, has been used for the past 12 or 15 years. The wood was heated to a temperature of about 85° under a pressure of about 90 lbs. It has never been found that this treatment injured the fiber of the wood in the least. The speaker has seen the tests made by Professor Thurston and by others, and these tests showed that the timber was strengthened. The difficulty found with vulcanized wood was, that when it began to decay, that decay began at the surface, not at the heart. The sterilizing treatment was sufficient in that respect, and whenever decay began it was caused by contamination from outside, evidently by being washed by the rain water. Mr. Deghuée.

The process described by the author seems to be especially adapted to overcome that difficulty. It produces practically vulcanized wood with an antiseptic shell. It would seem that, as the heart of vulcanized wood never decays and an efficient antiseptic shell is provided, the process not only overcomes the troubles encountered with vulcanized timber, but also the objections to ordinary creosoted wood. In the latter, the objection is reversed, as the wood frequently decays at the heart on account of imperfect sterilization. Another objection is that it is soft, and does not resist the crushing strain. Creosote, although it may not be soluble, is washed out, perhaps by the mechanical action of the water. The speaker has experimented with wood treated by the process described in the paper, and has found that it actually shed water, the water running in drops on the surface. He has made soaking tests of the wood, and found that absolutely none of the material will dissolve. After soaking the wood for three or four weeks it was impossible to detect any of the preservative material in the water.

The term vulcanizing is probably only a trade name, given to the treatment on account of its being a heating process. In vulcanizing it has seemed to the speaker that the treatment is primarily a sterilizing process—simply a thorough sterilization of the wood—the secondary action being the decomposition of the sap and the production of a small quantity of wood tar creosote in the sap of the wood. This creosote is antiseptic, but the amount is rather small, and it has been found that it was not sufficient to prevent external contamination; that a fungus could grow in the wood if it could penetrate it from the surface, and that in time the surface would begin to decay. It is also probable that these antiseptic bodies which were produced by the heating were not strong enough to prevent that decay.

The theory that the elements of the sap were hardened and filled the pores with impervious substance has not been verified.

Frank W. Skinner, M. Am. Soc. C. E.—This paper gives an interesting description of a process which seems to be admirably adapted to secure the ends which are sought. Mr. Skinner.

Mr. Skinner. The principal requirements in connection with the process are: The seasoning of the wood and its sterilization at a high temperature, the extraction of the volatile substances in the timber, the operation of the vacuum for withdrawing the air, and injecting an antiseptic and waterproof substance. The question as to whether or not all these processes are desirable, are harmless or are beneficial, should be considered. The weight of evidence presented seems to be that the degree of heat maintained in seasoning and sterilizing is not to be considered as detrimental to the fiber or the strength of the timber.

The question as to whether or not the removal of all the volatile oils in the timber is detrimental has not yet been raised. The speaker doubts the advantage of removing them. He has positive assurances, from those whose investigations and judgment he greatly respects, that the extraction of all the juices of the timber impairs its strength and makes it brash. That point is worthy of discussion. Supposing that, however, to be settled in either way, we obtain a piece of timber which is supposed to have been sterilized; that is, admitting the decay of the timber to be due to bacterial agencies, the bacteria are all killed. After this the injection of the antiseptic material seems to be somewhat supererogatory. If the timber is to be filled with antiseptic material, why need it be sterilized first? If the material is an antiseptic, it should be an efficient one without sterilization, and it would seem that Mr. Kummer's formula for an antiseptic is likely to give very good results. The waterproofing and the constituents of the liquid seem to be desirable, and the speaker is of the opinion that it seems likely that the process should be preservative and increase the durability of the timber.

There are one or two points in reference to the manufacture which the author should make clear. In the first pressure treatment of the timber, is pneumatic pressure applied? Apparently, no liquid or water is introduced; so that it may be inferred that the first is an air process. Secondly, the schedule of time required for the treatment of the timber is stated to be twelve hours. Is that twelve hours of net time or of gross time? Can a charge of timber be brought from stock at 6 A. M. of a given day and be turned out as a finished product at 6 P. M. of the same day? If so, the process is materially more rapid than most of those with which we are familiar.

There is one point in the paper to which the speaker is obliged to take exception, viz., the statement that the impregnation with an antiseptic fluid is very difficult or impossible in the case of large timber. There is to-day established in Philadelphia a company which impregnates, to a hitherto undreamed-of extent, hard and soft timber of any merchantable size, 12 x 12 ins. or greater, right to the core. In that process a piece of timber is put in a retort and saturated to 120% in

from 40 minutes to three hours according to its thickness. A piece of white pine 1 in. thick can be saturated to 120% in 40 minutes, and a timber 10 ins. square is saturated to the same percentage in a little less than three hours. The treatment of hard wood requires about 40% more time. By 120% saturation it is meant that a stick of timber weighing 100 lbs. when put in the retort will weigh 220 lbs. when removed. This is effected in a closed retort by means of very heavy hydraulic pressure, about 500 lbs. for working pressure, and as high as 1 000 lbs. if necessary; and the apparatus is such that this pressure can be applied in a very few minutes. In a treatment, which the speaker witnessed, some pieces of timber were inserted in the retort, saturated to nearly 200% and taken out in 63 minutes, and the retort was ready for another charge. That is, a round trip was made in about an hour, so that the injection of antiseptic fluid can be made to the heart of timber of any size in length and cross-section. That process of complete impregnation, together with the formula and materials described in the paper, might lead to exceedingly valuable results. It might obviate the necessity of sterilization and possibly the necessity of high temperature in seasoning. Mr. Skinner.

The high-pressure process, just described, is not used for wood preservation, but for the purpose of making the wood non-inflammable. The speaker has made pine wood shavings $\frac{1}{64}$th of an inch in thickness, has placed a spoonful of them on a platinum plate and put that over a Bunsen burner and held it there in the fiercest flame for probably 10 minutes. They were not consumed; they were blackened and could be crumbled easily, but under no circumstances would they blaze. A block of wood 2 or 3 ins. thick, and thus treated, was placed in the furnace of a steam boiler and left there over night. In the morning it was charred on the outside, but had not been consumed.

Henry Goldmark, M. Am. Soc. C. E.—A comparison between the cost of the process described by the author and that of the improved chloride of zinc method would be interesting and instructive. There is little doubt that the injection of creosote preserves the timber longer, and under more unfavorable conditions, than any other process. It is, however, by far the most expensive means of protection. Mr. Goldmark.

The whole question of timber preservation is, after all, one of economy. The point is, not whether the ordinary or an improved creosote process is the best thing, but whether something else much cheaper will not, at least for railroad ties, serve a useful purpose. The German railroads have had an extended experience for many years with the Rutgers process. This consists in injecting chloride of zinc, with the addition of a small percentage of dead oil of tar. The theory of the method is, that the dead oil of tar will prevent the zinc salt, which is soluble in water, from being washed out.

The results, as given out by the company which does most of the

Mr. Goldmark. work, are extremely favorable. They are based on official investigations by the German government, and are believed to be reliable.

The cost of this improved zinc process is so much less than that of creosoting that it is more likely to be adopted on a large scale in this country than more expensive methods.

Mr. Cartwright. ROBERT CARTWRIGHT, M. Am. Soc. C. E.—The preservation of wood is very desirable, provided it can be accomplished at a minor expense. The speaker at one time burnettized a great deal of bridge timber. The process consisted of enclosing the timber in a tight receptacle and boiling out, as it were, the pyroligneous acid. Of course, being under pressure and in a water-tight, hermetically sealed tank, there was no opportunity for the oxygen to get at the acid. The hot water dissolved the acid and was then drawn off. Dry heat was then applied and a solution of sulphate of zinc was introduced under pressure and thereby injected into the tubes forming the wood fiber.

The speaker has frequently used a second time the clamps and keys from a worn-out bridge. The Indians frequently stole the ordinary timber, but would not touch the burnettized sticks, because they could not saw them nor split them, and the wood would not burn. The burnettizing process, however, was very expensive, and to-day the object is to get the best preservative for the least amount of money. It is said that the process described in the paper is economical, and this is a point in its favor.

Mr. Bogue. V. G. BOGUE, M. Am. Soc. C. E. (by letter).—Notwithstanding all the creditable work which has been done, we must confess, that, during the last 15 or 20 years, there has been but little absolute progress in the science of wood preservation. We know that the creosoting process is the most reliable, but no process has yet been discovered which can be fully depended upon in all situations and for all varieties or classes of timber.

The author states that the results obtained with creosote do not indicate that after treatment there often occurs such cracking or checking as to expose the untreated, heart portions of the timber to decay. This is contrary to the writer's observations. Only recently, in making some railroad examinations, he has found that, in the majority of instances, where creosoted piles had decayed, the trouble was clearly caused by checks which had occurred after the structures had been built. If the sterilizing process will quite fully develop the checks, without working damage to the timber, so that when the creosote is turned on it will reach all portions of the material which the air would ever be likely to reach, we may have in Mr. Kummer's suggestion a distinct advance. The writer would like to see the suggestion tried on a scale large enough to prove its value.

Mr. Osgood. JOSEPH O. OSGOOD, M. Am. Soc. C. E. (by letter).—This subject is an important one to all engineers who are called upon to protect

timber from the teredo or from decay, and has been treated by Mr. Kummer in a clear and interesting manner. Mr. Osgood.

If ties could be used without driving spikes or boring holes in them, the method of treatment suggested would probably prove more effectual than under present conditions of service.

If the penetration of the antiseptic material is only about 1 in. from the sides of the tie and 6 to 8 ins. at the ends, only half the tie is treated at the section where the spike holes occur, and decay at those points is not long delayed.

It is noticeable that a good quality of long-leaf yellow pine has been selected for the experiments. The heart of this timber is full of resin and is fairly durable without treatment. It is said, however, by those who make a business of treating timber, to be much less suitable for the purpose than the woods of more porous structure, and which, untreated, decay much sooner, but which will take up a much larger amount of the creosote or other treating material.

It would seem to be entirely reasonable that the injection of resin with the creosote should make the surface harder, and might serve to keep the creosote from washing out, but it would tend to lessen the effectiveness of the creosote as an antiseptic, and it does not seem probable that it would be at all effective against the teredo, which eats the resinous heart of long-leaf timber with great avidity, and requires a very strong dose of creosote oil to affect its appetite.

It has been stated that all decay in timber begins at the outside of the stick. This statement would seem to be subject to some modification, as it is not uncommon to find oak sticks hard on the exterior surface and rotten inside; and even long-leaf yellow pine is frequently destroyed by what is known as "red heart," a fungous disease, in which the heart of the stick is most affected.

As to forcing creosote into long-leaf pine for protection against the teredo: this is now done effectively by the Galveston Wharf Company at Galveston, only creosoted piles being used in the main wharves of that company, and these, the writer is informed, give satisfaction. It is said that these are impregnated with the heavier oils of coal-tar to the extent of from 20 to upwards of 25 lbs. per cubic foot, and the writer is informed by the Manager of the Creosoting Company which treated the piles for the present pile bridge of the Gulf, Colorado & Santa Fé R. R., across Galveston Bay, built a number of years ago, that 27 lbs. of heavy dead oil per cubic foot was used in them, or in as many of them as would take that amount.

Long-leaf yellow pine, when thoroughly creosoted, generally shows little of the heart wood which is not distinctly colored by the oil, although, in the piles coming under the writer's observation, there has always been some untreated heart. The timber appears to be much weakened by the process, and thoroughly creosoted piles will some-

Mr. Osgood. times break in handling, under circumstances which would not injure an untreated stick of the same timber. The timber seems to be very brittle. The great objection to creosoted timber at present is the expense. Inquiry, three years ago, when several thousand piles were required in a bulkhead to retain a filled wharf at Sabine, Tex., showed that the piles, which then cost about 6 cents per lineal foot delivered, would have cost over 40 cents per foot, if creosoted.

In treating piles great expense is caused by having to treat the whole length. Treatment is undesirable for the portion in the ground, where the teredo does not work, and is usually unnecessary above the water, so that it frequently happens that more than half the length of the pile is treated without any special benefit. Various ways of overcoming this difficulty, and of using creosote without distributing it where not needed, have been suggested, and some of them tried. Experiments are now being made with piles which have a $1\frac{1}{2}$ or 2-in. hole bored through the center of each stick from the top to a little below the surface of the water, these holes being filled with creosote oil from time to time. The oil gradually distributes itself outward and downward, and, if this goes on rapidly and thoroughly enough, will, it is argued, reach the exposed surfaces of the piles and prevent the ravages of the teredo.

An attempt has also been made to stop the progress of the teredo, in piles already slightly affected, by boring a slanting hole at about mean low water and reaching down as far as possible without going through the stick. These holes, of 1 or $1\frac{1}{4}$ ins. diameter, are filled with creosote oil at intervals as the tide permits and as the oil soaks away. The holes are kept plugged at the top to prevent waste. It is too early to determine whether either of these experiments will prove effective or of any value.

A writer in one of the engineering periodicals not long since called attention to a method of forcing liquids into timber through a cap on one end of the stick, which permitted a pressure to be applied to force the liquid lengthwise through the pores of the wood. This would seem to offer advantages in treating piles, as the process could be stopped when any portion of the length had been saturated, and probably a hole bored in the middle of the stick, nearly as far down as it was desired to distribute the oil, would be of service in this connection.

Mr. Whinery. Samuel Whinery, M. Am. Soc. C. E. (by letter).—In discussing any method for the preservation of timber, the conditions under which the wood is to be used must be considered, and the character and extent of the preservative process should be suited to resist the causes of disintegration to which the wood will be subjected.

When exposed to brackish water, wood must be protected against the ravages, both of ordinary decay and of the teredo. When used for railroad ties, and generally for structures on land, it is necessary to

make provision against ordinary decay only. For the latter purpose it would seem that any process which, like that under consideration, first thoroughly sterilizes the timber throughout its whole mass, and then fills the cells to a greater or less depth from the surface with a permanent material which will exclude air and moisture, would prove efficient. It is not even necessary that the sealing material should penetrate the wood at all if the surface can be made impervious to air and moisture and maintained in that condition. If these assumptions are well founded, wood-preserving processes may cover a wide latitude, and there should yet be plenty of room for invention. The modified process of creosoting wood described by Mr. Kummer is rational, and should be efficient, particularly in preserving railroad ties. Mr. Whinery.

Those who have had occasion to work in the yellow pine forests of the Southern States know that the endurance of the "pine knots" is proverbial. When the public lands of those States were originally surveyed by the Government, these pine knots were frequently used for corner stakes or monuments, and they have often been found in place in a perfect state of preservation after being in the ground for more than thirty years. The principal peculiarity of these knots is that they are often perfectly saturated with pine resin, and their practical indestructibility is, without doubt, due to that fact. The process under discussion seems to be an approach to this wood-preserving process of Nature.

Horace J. Howe, M. Am. Soc. C. E. (by letter).—Referring to the tie question and the apparently prohibitory cost of treating the whole tie, it should be borne in mind that the life of the tie is the life of the portions adjoining the rails, and that in first-class, well drained ballast it is the top half of the stick at those points which suffers most, and necessitates renewal. Cannot a method of treating them be devised under these conditions? Mr. Howe.

In looking over piles of old ties taken from the track, one is struck with the fair condition, in most cases, of the middle portion of the tie, say for 4 ft. in length. The ends are split, owing to various causes, one of which is that trackmen, in originally placing the new tie, are allowed to use picks freely. Thus the ends are split off, and, naturally, cracks develop and rot sets in near the rails.

The heart of the tie being on the under side requires less attention than the top. Let the tie be dapped under each rail for 12 ins. in length, and let four spike holes at each end be bored by template for the pattern of rail to be used. It is stated by W. W. Curtis,* M. Am. Soc. C. E., that this can be done, in connection with the treatment, at a cost of about 3 cents per tie. Could not each tie be treated at these two places for a total length of 2 ft., with or without an isolating chamber?

* *Transactions*, Am. Soc. C. E., Vol. xlii, page 373.

Mr. Howe. The holes would distribute the preservative to the points where it was most needed, and the waterproof quality of Mr. Kummer's mixture, for instance, would take effect on top.

It looks as if the expense might be reduced considerably, say one-third, in comparison with the corresponding treatment of the whole tie.

It might be said that the tie would be unfavorably affected by difference in homogeneity, but this would have to be determined exactly by experiment.

Mr. Moncure. WILLIAM MONCURE, M. Am. Soc. C. E. (by letter).—The preservation of timber is a subject which has been discussed extensively, but, as yet, there seems to be no method by which the cost is brought within practical limits. The cost of the method proposed by Mr. Kummer is not given.

In 1886, in the construction of one of the southern railroads, all the signal and mile-posts were made of long-leaf North Carolina pine, the entire log being used, without any regard to the quantity of sap or heart. These posts were 4 x 6 ins., and the part above ground was painted with three coats of white lead in linseed oil. The lower part, up to a point just above the surface of the ground, was placed for 15 or 20 minutes in an open kettle containing 50 % of pine tar and 50% of resin, the mixture being kept above the boiling point. These posts are standing now, and when examined two years ago showed no signs of decay. The cost of this process did not exceed 15 cents for each post. It is possible that the wood was thoroughly dried by the heat, and that the tar and resin filled the outer pores, leaving also a coat outside which prevented the entrance of moisture.

The oak tie, without doubt, is best suited for roads where there is heavy traffic and much curvature, and seems to be the most difficult to preserve by the creosoting or any similar process. Investigations seem to have been directed almost entirely to creosoting or similar processes, and, so far, seemingly without practical results. It is possible that if investigations were made in the direction of drying the timber and coating the outside with a substance which would prevent the entrance of moisture, some method might be found which would bring the cost within a practical limit. There is a cypress in some of the Southern States which has been used for ties and has lasted for 25 years, but cypress is objectionable because the wood is soft; the rail cuts the tie, and the spikes do not stay in place, thus causing the track to get out of gauge. These objections can be overcome by the proper use of tie-plates, and a cypress tie with tie-plates ought to especially recommend itself to roads on which the curvature, gradients and traffic are light.

The question of obtaining ties is at present of great importance to the maintenance-of-way departments of railroads, and these notes

are presented with the hope that this subject will be continually kept in view and result in developing some practical method which will be within the means of any railroad. Mr. Moncure.

J. I. Boggs, Assoc. M. Am. Soc. C. E. (by letter).—In discussing the paper by Walter W. Curtis, M. Am. Soc. C. E., the writer gave the results of some observations on creosoted pine ties along the line of the Houston and Texas Central Railway.* These ties had seen 20 years' service, and it was estimated that fully 80% of the original number was yet in the track. The objections to them were: First, the soft wood used, which permitted the rail to cut into the timber very badly; Second, their high price. Mr. Boggs.

These observations were recently confirmed by M. G. Howe, M. Am. Soc. C. E., the present Engineer of Maintenance-of-Way, and by G. A. Quinlan, M. Am. Soc. C. E., the General Manager of that road, in a late issue of *Engineering News*. Twenty years ago Bethell's process was still in its infancy, and, owing to its excessive cost, managers hesitated to use it, except to a very limited extent; but with the facts deduced from our past experience, certainly none can justly say that "no process has yet been discovered which can be fully depended upon" for the preservation of timber, and it appears to the writer that the time for hesitation is past, especially when we consider the depleted condition of our forests.

The writer has seen some rotten creosoted piling, but in nearly every case this was caused by the bridge carpenters cutting off the ends of the sticks after the piling was driven and neglecting to apply the hot preservative to the exposed end, thereby permitting it to absorb moisture from the atmosphere for days and sometimes weeks at a time. In other cases this decay may be attributed to the treatment of defective timber.

The writer has never been troubled with rot caused by the checking of the creosoted pine timbers after they were in the structure.

It is unfortunate that a great many of our co-workers imagined the poorest class of timber to be the best for creosoting purposes, because it has given rise to the fallacy that "the timber is much weakened by the process." The fact is, if the timber is properly treated, the dead oil neither adds to nor detracts from its strength; but, in nearly every case, comparisons are made between the very poorest and the very best material, the result being the hastily and erroneously drawn conclusion that "the process weakens" it. The writer has seen "thoroughly creosoted piles sometimes break in handling," but close investigation in each instance has shown that originally they were of the poorest pine, such as is known as the "Loblolly"; that they were stacked in the yard to "dry out" for two or three months before being treated, and were, in fact, rotten before the oil ever touched them.

* *Transactions*, Am. Soc. C. E., Vol. xlii, p. 348.

Mr. Boggs. R. Montfort, M. Am. Soc. C. E., Chief Engineer of the Louisville and Nashville Railway, is in a better position than anyone known to the writer to speak with authority on the action of the teredo on creosoted piles, as his line traverses a number of salt water inlets along the Gulf Coast, and probably owns one of the oldest creosoting plants in the country. It is the writer's impression that he has entirely abandoned treating his piling, and is now protecting it with salt-glazed sewer pipe, as creosoting did not protect against the teredo. The Gulf, Colorado and Santa Fé Railway had a similar experience at Galveston, Tex. About the year 1881 they built a creosoted pile trestle, about 2½ miles long, across Galveston Bay. It lasted about 2 years, and was then rebuilt with elm, which, of course, required constant renewals, at a cost of about $1 000 per month. This continued until 1897, when they rebuilt with creosoted pine, with what results the writer is not in a position to say.

The writer once experimented on pine poles containing nothing but sap. The poles were cut in the spring and completely immersed in fresh water. The following spring they were found to have turned into "lightwood," that is, heavy, resinous, pitch pine. They were then placed in salt water, and, up to the following fall, the teredo had failed to attack them to any material extent; hence the writer believes the use of a percentage of resin in the processes of either Bethell or Burnett would be very beneficial; but, that the dead oil of coal-tar will ever alone protect against the teredo, is, in the light of past events, very doubtful.

The prevailing custom of subjecting only the poorest class of timber to the creosoting process is to be greatly deplored, and when a better material is more extensively used very high results may be confidently expected. The use of very high temperatures will unquestionably kill the germs, but, unfortunately, will kill the fiber of the timber also.

Mr. Curtis. Walter W. Curtis, M. Am. Soc. C. E. (by letter).—The engineering profession should be ready at all times to give a hearing to the advocates of any new processes affecting the materials of construction; and to test such processes when it is possible to do so without too great expense; or when the possible resulting failure can be determined before the expenditure has become very large.

There is now, and will be in the future to a greater extent, a strong desire to experiment with new wood preservatives. The two methods now in use are old. The evidence of their value, in the writer's opinion, is sufficient to justify their general adoption, but is not as complete as could be desired, and, from the nature of things, is not likely to be. The difficulty of keeping tie records on long lengths of road for 15 and 20-year periods is such that those who desire to find fault with them will always have opportunity. One of the greatest

dangers to the cause of timber treatment in this country is likely to be this desire for something new—and cheap. Mr. Curtis.

In the writer's study of the question, it has been very amusing and instructive to read the circulars and endorsements therein, of 20 years ago, of methods long since gone by the board. It seems almost necessary to demand of the advocates of every new treatment, that it should first be tested for 10 or 15 years and then presented to the attention of the public. Certain it is that nothing short of such a test can be considered as conclusive, as to comparative value, in the case of ties.

Evidently, however, all engineers do not take quite such a pessimistic view of the matter. A committee of one of the Associations of Engineers of Maintenance-of-Way recently reported very strongly recommending the general adoption of a method of treatment of ties by painting them with or dipping them in a certain preservative compound; the advice being prefaced with and based upon a statement that by such treatment a life of 18 years could be secured from timber naturally having a life of but 6 years. The evidence to support such claims is very small and entirely insufficient to warrant any such general recommendations. In the advertising matter regarding one of the most recent processes offered the public, is the report of a learned professor, in which it is asserted that a life of from 13 to 15 years can be secured from soft wood ties treated by this particular method; and the evidence on which this is based is that poles so treated and placed in soil favorable to decay were sound after $2\frac{1}{2}$ years.

The writer does not intend to decry either of these processes; they may be of great value, but the evidence thereof, as applied to ties, is extremely limited.

Mr. Kummer describes a treatment intended to overcome certain mechanical difficulties "in connection with creosoting, particularly railroad ties." Evidently, he refers to the fact that creosoting does not harden timber and that the usual allowance for ties, 10 lbs. of creosote per cubic foot, does not completely saturate the wood. This is undoubtedly true, but it seems hardly fair to condemn creosoted ties therefor, without some more definite data as to actual bad results. The recorded history of creosoted ties in the United States is not very great, but, so far as it goes, does not indicate that the deterioration from rail cutting is so serious as to detract greatly from the value of the treatment.

If Mr. Kummer has shown a means whereby such difficulties as do exist, however great or small they may be, can be removed, he has earned the thanks of the profession; but we must be careful lest, in running from the troubles of which we know, we do not also leave good qualities of which we are certain. As the paper did not state the comparative costs of the creo-resinate treatment and ordinary creosoting,

Mr. Curtis. the writer inquired of the company operating the process, and was told that the costs were the same, using equal amounts of preservative material in each case. For a tie creosoted with 9 to 10 lbs. per cubic foot the English records show a life of from 16 to 20 years; using a rail supported in a cast chair. If creosoted ties here are provided with tie-plates, at an additional cost of about 10 cents, there is every reason to expect as long service; indeed, Mr. Boggs, in his discussion* of the writer's paper, states that on the Houston and Texas Central R. R. 80% of the creosoted ties laid there in 1877 were still in service after 20 years, and that without a tie-plate.

The author's remarks as to the necessity for complete sterilization in creosoting are no doubt correct. In this respect the chloride of zinc treatment has an advantage, for, while sterilization there is probably advantageous, the use of heat is primarily to so season the wood and open the ducts, as to permit the entrance of the preservative fluid. This completely saturates the timber, if properly done, and the preserving and toxic agent is deposited throughout all parts of the timber. There is a decided lack of data as to the penetration of heat into ties during steaming, and the figures given in the paper are welcome and pleasing as well, demonstrating, as they do, the thoroughness of the steaming in Burnettizing. Mr. Kummer states, "with a temperature of 230° Fahr. in the cylinder, 178° was secured at the center (of the tie) in 2 hours, while with 290° at the outside, 249° was secured in 2 hours." Presumably, the 2 hours referred to is the time interval between the admission and exclusion of steam to and from the cylinder. In Burnettizing, it is customary to steam for at least 3 hours, the temperature being from 250° to 260° Fahr. The mean of the two cylinder temperatures given by Mr. Kummer is 260°, and the mean of the resulting tie temperatures is 213½°; so it is reasonable to conclude that the usual practice in Burnettizing results in complete sterilization. With reference to the use of a higher temperature under pressure, without injury to the timber, the writer has no data. He is not satisfied with any explanation of the philosophy of it, known to him, but the evidence seems to support the claim. The danger to be feared is not so much the checking of the wood, as the breaking down and disintegration of the fiber. It would be very interesting to know the relative absorption of preservative by timber treated according to the author's method and by the use of heat of 250° for the same length of time without pressure.

Table No. 1 shows the amount of absorption of preservative in green and in vulcanized timber. It is well known that it is impossible to inject any large amount of anything, even water, with the usual pressures and time of application, into timber which has not first been prepared for it by steaming. Before the value of vulcaniz-

* *Transactions*, Am. Soc. C. E., Vol. xlii, p. 348.

ing can be admitted as a preparatory treatment, it must be shown that steaming at the usual heat would not secure the same results. Even if the time of such steaming with 250° temperature was one or two hours longer, in the writer's opinion, it would be much safer. Mr. Curtis.

Table No. 2 shows the uniformity of absorption—or lack thereof. It will be noted that the range is from 5.62 to 16.87 lbs. per cubic foot; the variation being just 200% of the lower extreme. Evidently, vulcanizing is not successful in entirely eliminating the idiosyncrasies of wood.

Mr. Kummer states that thorough sterilization is not generally secured during creosoting treatment. The writer understands that creosoting works generally use as high temperatures in steaming as do Burnettizing works, and continue it for as long or longer periods. What evidence has the author that the temperature of 212° at the center of the tie is not secured in creosoting?

The use of the creo-resinate process is recommended by the author for marine work. The writer recently received a letter from Mr. J. W. Byrnes, Superintendent of the timber treating works at Beaumont, Tex., whose experience in this line extends back some 30 years.

He states that Mr. J. W. Putnam made a test of a combination of resin and creosote, while operating the works at Pascagoula.

"He treated a quantity of sheet piling for the Galveston Wharf Company, about 10 years ago, and in less than 2 years the teredo had completely destroyed the wood. The President of the Wharf Company appointed me as the arbitrator in their behalf, and I recommended that Mr. Putnam be allowed to act in his own behalf. We had the piling cut out at the point of greatest exposure and divided them into short cuts and split them in small sections. The wood showed a perfect saturation, but, while only one-third resin had been used to two-thirds oil, there was no odor or sign, other than color, that would indicate that a particle of dead oil had been used. The resin had totally destroyed the quality of the oil as a preservative against the teredo. About 1869, one of the leading chemists and physicians of New Orleans conceived the idea of treating wood with a mixture of resin-oil and resin, for marine use. We were at that time distilling resin-oil in connection with our coal-tar distilling works. We treated a number of samples of 4 x 4-in. pieces which he had fastened to piling or other structures at Lake Pontchartrain, Biloxi, Mississippi Sound, and Fort Phillip and Fort Jackson, at the mouth of the Mississippi River. The treatment seemed to give the wood a relish for the teredo. The Biloxi test was destroyed totally within 2 years. The Lake Pontchartrain and mouth of river samples held out for a year or two longer, but none to a greater extent of time than the natural wood. While the treatment of wood with a mixture of dead oil and resin may be effective as a preservative against decay, it has been positively proven by thorough tests that no good results will be obtained when used for marine work."

The process described by the author differs somewhat from that followed by Mr. Putnam; and it may be that the differences are sufficient to secure a more happy result. As a matter of history, however, the foregoing should be recorded.

Mr. Curtis. Mr. Byrnes is strongly of the opinion that, to get satisfactory results from creosoted timber in marine work, it must be allowed to dry out for from 30 to 60 days after treating, when the creosote in the outer part of the timber becomes thick and gummy. If this is done, and the treatment has been properly given, he is satisfied that no difficulty will be experienced with the washing out of the creosote; and the durability of the work will prove satisfactory.

It is to be hoped that future experience with the creo-resinate treatment will demonstrate its value to be as great as the author anticipates. The writer, however, is less interested in improving upon creosoting practice than in approximating the results secured thereby at a less cost. The great need of the railroads to-day is to demonstrate or confirm the value of the zinc-chloride treatment; and to improve thereon.

Mr. Montfort. R. MONTFORT, M. Am. Soc. C. E.—The writer's attention has been called to Mr. Boggs' discussion, in which the following statement is made:

"R. Montfort, M. Am. Soc. C. E., Chief Engineer of the Louisville and Nashville Railway, is in a better position than any one known to the writer to speak with authority on the action of the teredo on creosoted piles, as his line traverses a number of salt water inlets along the Gulf Coast, and probably owns one of the oldest creosoting plants in the country. It is the writer's impression that he has entirely abandoned treating his piling, and is now protecting it with salt-glazed sewer pipe, as creosoting did not protect against the teredo."

Mr. Boggs is mistaken in his impression. The Louisville and Nashville Railroad Co., during the past twelve months, constructed a large dock and warehouse at the foot of Commendencia Street, Pensacola, Florida, in which some 6 000 creosoted piles, averaging 70 ft. in length, and treated with 20 lbs. of creosote oil per cubic foot, were used. In addition to this, during the same period, about 2 000 000 ft. of sawed timber were used by this company on the wharf referred to, and for bridges, culverts, etc. It certainly, therefore, cannot be said that the Louisville and Nashville Railroad Company has entirely abandoned the use of creosote oil for protecting piling and timber against the teredo and against rot.

It is true that after a service of 14 years the creosoted piles in trestles across Biloxi Bay, and Bay St. Louis, on the New Orleans and Mobile Division, and across Escambia Bay, on the Pensacola and Atlantic Division of the Louisville and Nashville Railroad, were found to be attacked to a more or less serious degree by the teredo, and the piles in these trestles were further protected against the teredo by concrete or by vitrified clay pipe as described in a paper by the writer, read before this Society November 1st, 1893.* The same creosoted

* *Transactions*, Vol. xxxi, p. 221.

piling, driven in 1878, namely: 2 626 piles at Bay St. Louis, and 1 648 piles at Biloxi Bay, or a total of 4 274 piles, with the exception of 3 that have been replaced by new piles on account of rot, and 122 piles that were destroyed by the hurricane of October 2d, 1893, are still in use, giving good service, and with prospect of lasting several years to come. Untreated piles at these bridges would not last, on an average, more than 7 years as against rot, or 6 months against the teredo. Mr. Montfort.

Mr. Boggs does not state as to the treatment adopted, or the quality of the oil used in the piles of the Gulf, Colorado and Santa Fé Ry., at Galveston, Texas, which gave such poor results. From the writer's experience he is inclined to think there must have been some defect, either in the treatment or in the quality or quantity of the oil used that caused such results.

The writer believes Mr. George S. Valentine, who for years has been in charge of The Eppinger and Russell Company's creosote works, Long Island City, N. Y., treated piles in the early 70's that were driven in Galveston Bay, and are still doing service if they have not been destroyed by the recent hurricane.

In discussing the paper referred to, the writer said:*

"In recent years, we have been experimenting with a mixture of resin and dead oil, with the hope that the resin will aid in fixing the oil and prevent it from being dissolved or washed out. Sufficient time has not elapsed to form any definite conclusion, although, so far, piles thus treated have given satisfactory results."

It is to be regretted that further experience with piles treated with dead oil of coal-tar and resin has not given the satisfactory results hoped for. In 1890, it having been found that the teredo had commenced to attack the piles in piers of East and West Pascagoula River bridges, driven in 1876, it was decided to strengthen the piers by replacing two piles at each end of each pier, consisting of sixteen piles. These piles were 80 ft. long, 10 ins. in diameter at the small end, and were driven 40 ft. into the bottom of the river. They were treated with a mixture of resin and oil in the proportion of 45 barrels of resin to 30 barrels of creosote oil, which is about the same proportion Mr. Kummer proposes to use. The total number of piles treated in this way and driven at these bridges was 88.

A recent examination of these piles for rot, by boring, showed that of the inside piles driven in 1876, and therefore 24 years old, only 8% showed rot in the heart, while of those treated with the mixture of oil and resin, driven in 1890, and only 10 years old, 43% showed rot in the heart. An examination by diver showed that the piles 24 years old, originally treated with creosote oil, were in better condition than the piles only 10 years old treated with the creosote oil and resin mixture.

From this it will be seen that the idea of mixing resin with the creosote oil did not originate with Mr. Kummer, but was used by the

Transactions, Vol. xxxi, p. 246.

Mr. Montfort. late J. W. Putnam, Assoc. Am. Soc. C. E., who did the work referred to, under contract, more than 10 years ago.

The writer hopes that Mr. Kummer's experience with piles so treated will be better than the results obtained on the Louisville and Nashville Railroad.

Mr. Felt. C. F. W. FELT, M. Am. Soc. C. E. (by letter).—Some of Mr. Boggs' statements in reference to the bridge of the Gulf, Colorado and Santa Fé Railway Company across Galveston Bay are inaccurate. The facts in relation to the original Bay bridge are as follows:

This bridge (about 2 miles long) was built in 1875. The contract for the treatment of the piles specified the use of 8 lbs. of creosote per cubic foot. As nearly as the writer can ascertain, the treatment was not carefully done, and was not uniform. Mr. Byrnes, who had charge of part of it, has stated that some of the piles were treated with a much heavier amount than that specified in the contract. Mr. B. M. Temple, then Chief Engineer of the road, reported seven years later that about 50% of the piles had been destroyed; and in 1895, when the bridge was removed, about 5% of the original piles were still intact.

In 1895 the bridge was rebuilt. The piles were of long-leaf pine, treated with 24 lbs. of creosote to the cubic foot, and at this writing they have not been attacked by the teredo.

Mr. Valentine. GEORGE S. VALENTINE, Esq. (by letter).—In May, 1874, in conjunction with Mr. R. S. Trundy, the writer built a small creosoting plant at Galveston to treat paving blocks, which were afterward laid on Market street, in that city. The lumber used was the ordinary sound and square-edged yellow pine, which was treated with 12 lbs. of dead oil of coal-tar per cubic foot, and laid under the Stow patent, which consisted in a wedge being driven between each row of blocks for half the depth of the block. The blocks were 6 ins. deep and 4 ins. wide. The wedge was 6 ins. long and 1 in. thick, and was driven into the ground 3 ins. This left between each row of blocks a groove 1 in. wide and 3 ins. deep, which was filled with fine gravel and a mixture of asphalt. This pavement is still in good condition. A few years ago, when the motive power of the railroad was being changed from mules to electricity, the contractor, Mr. J. W. Byrnes, of that city, offered a reward of $50 to anyone who could find an unsound block among those which were taken up, and, although a great many persons were searching, he did not have to pay the reward.

Before the writer left Galveston, late in the fall of 1874, the Galveston Wharf Company and the Galveston, Houston and Henderson Railroad were negotiating with him for some creosoted piles. These piles were treated at our works by J. W. Byrnes of Galveston. Some of them received a light treatment and some a heavy treatment. Those receiving the light treatment were not a success, but those treated with 20 lbs. of dead oil of coal-tar per cubic foot are now in

use, and some which were taken up a few years ago showed no sign of decay or the effects of the teredo, although that insect will destroy an untreated pile in less than one year. Mr. Valentine.

In regard to the creosote and resin process, the late J. W. Putnam, of New Orleans, who was an authority on wood preserving, took out two patents for such a process, Nos. 405 907 and 405 908, dated June 25th, 1899. He had been experimenting with these compounds a long time before he gave it up some years ago, pronouncing it a failure.

The writer has made a large number of experiments with various compounds in order to find a material which would take the place of dead oil of coal-tar and cheapen the cost, but, so far, has been unsuccessful.

F. A. Kummer, Jun. Am. Soc. C. E. (by letter).—Mr. Curtis rightly says that the objections to creosoting, at present existing, are not sufficient to condemn it, and the writer certainly had no intention of making such a statement. On the contrary, the advantages of creosoting are such as to very generally recommend it, and Mr. Octave Chanute's very interesting paper* goes to show that creosoting, either in its usual, or in a modified form, is fast supplanting all other processes in European practice. Its disadvantages, however, are quite generally known, and therefore it would seem to the writer that any attempt to remove them by improving the process is a step in the right direction, and in this he feels sure that Mr. Curtis will agree with him. Mr. Kummer.

As for the recorded history of creosoting in the United States not showing that rail cutting is a serious obstacle to its general use, the writer must disagree with Mr. Curtis entirely, so far as eastern roads or those having heavy equipment and traffic, are concerned. The Manhattan Elevated Railroad published a report, during the management of the late Colonel Hain, in which it was directly stated that creosoting decreased the spike-holding qualities of the tie and increased rail cutting, and substantially stated that vulcanizing increased spike holding and decreased rail cutting. In England, the rail and chair construction, and the lighter equipment, does not make this question so vital. Tie-plates at considerable expense would help to remove the trouble in this country, so far as rail cutting is concerned, but not with reference to spike holding. This is the position taken by the Lehigh Valley Railroad, as records of discussions before the New York Railroad Club will show, and this road should be in a position to speak authoritatively, as they for some time owned and operated a creosoting works at Perth Amboy, now no longer used for that purpose.

The fact that creosoted ties have lasted without serious rail cutting,

* "The Preservation of Railway Ties in Europe," presented to the Society, October 17th, 1900.

Mr. Kummer. on the Texas Central Railroad for 20 years, does not by any means indicate that they would do this on the New York Central or on the Pennsylvania. The question seems to turn largely on the character of the timber used. In the East the great scarcity of chestnut and white oak is driving the eastern roads into the use of long-leaf yellow pine, a class of timber not readily treated by any method of preservation, and especially difficult when subjected to treatment by any of the processes in which water is used for carrying the antiseptic salt into the timber, owing to the highly waterproof nature of the natural pine resins. The great and increasing use of yellow pine by eastern roads, makes the treatment of that timber a desideratum. A gentleman, whose experience in the use of the Burnettizing process has been of the widest, recently stated that it is not practicable to satisfactorily Burnettize resinous long-leaf yellow pine at reasonable cost. The use of softer woods, with tie-plates, is no doubt readily possible, but outside of the question of crushing comes the equally serious question of the spike-holding qualities of the wood. It is the writer's opinion that while retaining all the advantages of creosote as a preservative, advantages which have led to the almost universal discarding of all other antiseptics in its favor in England and on the continent, the creosote-resin process very greatly increases the spike-holding qualities of the timber, while at the same time greatly increasing its resistance to crushing. These are mechanical questions and should be readily susceptible of proof. The experiments and tests which have been conducted and are now being conducted by the United States Wood Preserving Company at Perth Amboy and elsewhere, demonstrate this. If these results are not accepted, the writer feels sure that that company will not only be ready to furnish samples at any time for independent testing and investigation, but will place its plant at the disposal of the Society, or its members, at any time, with every opportunity for the making of experiments to demonstrate, not only their claims as to resistance to crushing, etc., but as to the complete sterilization of the timber, the absorption of the preservative fluid per cubic foot secured, and the length of time required. The writer makes this statement on the authority of the United States Wood Preserving Company, with which he is in no way connected.

The writer is glad to find that Mr. Curtis agrees with him in the necessity for complete sterilization. If such sterilization is generally secured in the Burnettizing process, so much the better for that process—but the figures given by Mr. Curtis do not demonstrate it completely. It is more difficult to secure a sterilizing temperature at the heart of a stick by steaming than by using dry heat under pressure. Experiments have shown that the penetration of heat is more rapid when wood is treated in this manner than when treated with steam at the same temperature—the explanation of this being that the water

condensing in the outer pores of the wood, and being a very bad conductor of heat, prevents the temperature at the center of the wood from rising as rapidly as it would otherwise. On the other hand, in the treatment by means of dry air, the hot air, being under pressure, is forced into the wood and assists in causing the temperature to rise. When these facts are considered, Mr. Curtis's conclusions, drawn from the figures in the paper, are not convincing. It is true that when these figures are averaged an outside temperature of 250° would give a temperature of 213½° at the center of the tie, but this is with hot dry air under pressure. If steam at 250° were used, this temperature at the center would probably not be obtained except at great loss of time. To use a higher temperature than this in the case of steam, or to keep the wood at this temperature for a much longer time, would injure the fiber of the wood. Mr. Kummer.

The whole idea of "vulcanizing," so-called, was to sterilize large timbers without injury to the fiber, by the use of hot dry air under heavy pressure. It has been intimated that vulcanizing caused no injury to the fiber of the wood, or impaired its strength, and this opinion is fully borne out by tests on vulcanized wood, made by Professor Thurston and others. The radical difference between the two methods of sterilizing, steaming and vulcanizing, lies in the use in the latter process of air pressure up to 90 or 100 lbs. to fully prevent ebullition or boiling under the high temperature used. It is the writer's opinion that unless sufficient pressure can be used to prevent boiling, it is not safe to subject timber to temperatures sufficiently high to produce complete sterilization in a reasonably short time. Mr. Curtis says:

"It is well known that it is impossible to inject any large amount of anything, even water, with the usual pressures and time of application, into timber which has not first been prepared for it by steaming."

This is certainly not a fact, since the company operating the creoresinate process is doing this very thing daily on large contracts. Mr. Curtis also says:

"Before the value of vulcanizing can be admitted as a preparatory treatment, it must be shown that steaming at the usual heat would not secure the same results."

Why must this be shown? Even if steaming at the usual heat would secure absolute sterilization in a reasonable time without injury to the fiber—which the writer does not believe—what advantage has steaming over vulcanizing in the treatment of yellow pine timber?

Mr. Curtis is quite right in saying that vulcanizing as a preparatory treatment does not eliminate the idiosyncracies of timber, and, of course, this is not claimed for it, or indeed for any other reputable process. It is only those processes, new or otherwise, for which perfection is claimed, that really tend to throw the whole subject of wood preservation into disrepute.

Mr. Kummer. The only evidence the writer has that the steaming temperature generally used in creosoting does not in the time allowed give absolute sterilization is the demonstration afforded by experiments conducted with self-registering thermometers to determine this fact. The writer has believed for some time that the great importance of complete sterilization is generally overlooked; but if Mr. Curtis has any evidence to the contrary, he is very glad to know it. The experiments were not made for the purpose of casting any discredit upon creosoting as generally done, or in fact upon any other process, but merely to insure the absolute certainty of sterilization in the particular process which formed the subject of the paper.

With reference to the use of the process for marine work, there is no scientific basis for the action of resin upon creosote as described by Mr. Curtis, and yet the results may be as described. The writer, however, believes that piles treated with creosote proper (and not with a mixture of creosote and resin as tried by Mr. Putnam) would have a much longer life if the washing out of the creosote oil could be prevented by a surface treatment of resinate of lime, as outlined in the paper. No claims are at present made for the use of creosote-resin as a protection against the teredo, although the writer understands that experiments are now being made to demonstrate its value in such service.

In view of Mr. Curtis's closing paragraph, that the great need of the railroads to-day is to demonstrate or confirm the value of the zinc-chloride treatment and to improve thereon—the following extracts from Mr. Chanute's paper* may prove interesting:

"*Great Britain.*—All antiseptics, save tar-oil, have now been abandoned in England, where 'creosoting' so-called, has grown to be recognized as the best process to use."

"*France.*—The latter substance (creosote) is now practically used exclusively by all the roads, save those of the State, which use a mixture of chloride of zinc and creosote, which will be more fully noticed when the German practice is described. 'Burnettizing,' *i. e.*, the injection of chloride of zinc alone has been extensively used, but is now abandoned, as the zinc is found to wash out in time, specially in moist situations."

"*Germany.*—Corrosive sublimate and sulphate of copper are now practically given up in Germany, and the State Railway also abandoned Burnettizing or the injection of chloride of zinc by itself in 1897. There are now three processes in use:

"1. Impregnation with chloride of zinc and tar-oil.

"2. Creosoting after seasoning and drying in ovens.

"3. Creosoting after desiccation in hot tar-oil."

The reference to creosoting, after seasoning and drying in ovens, does not indicate any great necessity for steaming, which Mr. Curtis regards as a *sine qua non*.

Mr. Chanute's objection to creosoting in the United States is its

* "The Preservation of Railway Ties in Europe."

cost, but there is no doubt that the conditions of service on heavily- Mr. Kummer.
traveled eastern roads, with frequent trains, heavy equipment and frequent bridges, tunnels and elevated structures, may justify a far greater cost per tie than would be justified on many of our great western systems, and this, too, independently of the actual first cost of the tie itself.

Mr. Montfort's interesting discussion throws some strong light upon the necessity of thorough sterilization. He cites instances of piles treated with creosote oil, and with creosote and resin mixed, in which heart rot was found to exist in a large proportion of the piles, especially in those treated with the resin mixture. There is no doubt that this heart rot could be prevented by thorough sterilization of the piles before treatment, and the writer is glad to note this strong evidence of the necessity of sterilization. It seems probable that the most perfect method for the treatment of piles would be with creosote oil alone, after thorough sterilization, the surface of the piles being subsequently subjected to a resin and lime treatment to form an insoluble outer covering to retain the dead oil. The conditions for resistance to the teredo and to ordinary decay are, of course, quite different.

The writer was not aware of Mr. Putnam's experiments with resin, creosote oil and petroleum, and certainly had no desire to claim any originality in the presentation of the creosote-resin process to which he was not justly entitled.

With regard to the zinc-tannin process, an interesting question arises. The impregnating solution, which is 90% water, is immediately followed by the tannin treatment, which latter, it is claimed, completely seals the pores against the entrance of moisture from the outside. If this be true, and moisture cannot enter from the outside, how can the moisture inside the timber escape, as is claimed it does? If, on the other hand, the pores are open to the escape of this large percentage of water from within, would they not be equally open to the entrance of water from the outside, and the consequent dissipation of the injected chemical?

Mr. Skinner's comments doubtless arise from a slight misapprehension. In saying that it is practically out of the question to force an antiseptic fluid entirely throughout large sticks of timber, this of course means "commercially out of the question." Doubtless it can be done, at great expense and loss of time.

With reference to the withdrawal of all the volatile oils, the process is designed especially to prevent this. The oils are valuable in the wood. Sterilizing alone would not preserve timber, because germs would enter from without. Complete impregnation is too expensive.

The writer regards the estimate of 10 years as the average life of un-

Mr. Kummer. treated yellow pine ties as too high. This might be the average life on elevated structures, but in the ground the records from several large systems indicate that the average life is about 7 years or less.

Mr. Chanute, in a paper before the Western Society of Engineers, recently said that he had been trying for years to find a solvent for metallic salts, such as zinc chloride, which would be oily in its nature, so as to prevent the preservative fluid from washing out, and that such a solvent would make a perfect process; but he had—at least at that time—never found it, and they are still generally using water in connection with the salts. The admixture of resin, creosote and formaldehyde gives an admirable combination of the antiseptic, waterproof, hardening and spike-holding qualities, in the writer's opinion, not equalled by the German combination of creosote oil and chloride of zinc.

This process, mentioned by Mr. Goldmark, is attracting a great deal of attention. The writer is convinced that it is a distinct advance over Burnettizing alone. How it would act if applied to yellow pine timber has probably not yet been determined, but Mr. Chanute is, the writer understands, investigating that question now. It would seem that its weak point might be in its application, particularly, to the inferior grades of timber, which cannot be used without a tie-plate, and which have low spike-holding qualities. It certainly could not harden the timber, and this the creosote-resin process does to a very great degree. In this connection, it is interesting to note that from 50 to 60% of the renewals on certain of our eastern lines with heavy traffic, come from crushing or failure of spikes to hold, and not from decay. For this reason inferior grades of timber, no matter how well preserved against decay, would not last longer than if absolutely untreated, unless the treatment were such as to very materially harden the wood and increase its spike-holding qualities. If a treatment which does this be used, and in addition be applied to timber such as yellow pine, which is in itself a hard and resisting timber, the results should so resist the effects of crushing and spike pulling that the preservative part of the treatment could come into play.

It is the writer's opinion that the creosote-resin process, with its positive waterproof qualities, its greatly increased hardness and spike-holding qualities, its thorough and complete sterilization, and the nature of the antiseptic used, is one of the best methods for the treatment of timber that has yet been advanced. The greatest argument against its general use would seem to be one of cost; but even that question is yearly becoming less important in those sections of the country where railroad travel is heaviest and traffic most congested.

Mr. Valentine's discussion is of great interest as indicating favorable results with treated wooden paving blocks, more favorable than those secured in many other instances in this country.

Mr. Kummer.

The patents referred to as issued to Mr. Putnam on a combination of creosote and resin, and which it is asserted Mr. Putnam declared a failure, must have differed very materially from that now used by the works at Perth Amboy, since the latter process has been granted full patents by the United States patent office. Whatever the process tried by Mr. Putnam was, it would be instructive to know the nature of the experiments which caused him to declare his process a failure. The indications are that they referred only to marine work, as a protection against the teredo, as mentioneed by Mr. Curtis in his discussion.

Mr. Valentine gives no reason why Mr. Putnam abandoned his compounds. It may have been due to inability to obtain proper penetration or possibly to the sticky nature of the product, such as is obtained when using creosote and resin alone. The attention of the writer has been called to a letter from the International Creosoting and Construction Company of Beaumont, Texas, in which the statement is made that the piles treated at Lake Pontchartrain by Mr. Putnam were "thoroughly saturated with resin oil, with a mixture of resin sufficient to form a body." Apparently, no creosote oil at all was used in this case. Of course, no claim is made that resin or resin oil alone will act as an efficient preservative.

Judging from the concluding paragraph of Mr. Valentine's discussion, he seems to infer that the object of the creo-resinate process is to cheapen the cost of treatment. This, of course, is a misconception of its purpose. The new process is a modified and improved creosoting process, yielding, it is claimed, a better and more durable product without increasing the cost. At present, the process is recommended principally for paving blocks and railroad timber, the question of its efficiency for use in piles being still unsettled, as already stated.

It appears that Mr. Putnam's treatment was confined entirely to piling. The results of his experiments, therefore, good or bad, even if his process was of a similar nature to that described in the paper, can have no weight in forming an opinion of its value for the purpose of preserving timber against decay.

The writer has made a careful examination of the patents issued to Mr. Joseph W. Putnam for a wood-preserving compound consisting of a mixture of resin, creosote and resin oil, and a mixture of resin, petroleum and resin oil. Mr. Valentine states that this process was abandoned. There are many reasons why such may have been the case, and still not throw the least discredit upon another process using apparently the same mixture. The mixture of resin and petroleum is undoubtedly inferior to that of resin and creosote, and may therefore be left out of consideration.

The patent papers say: "No particular proportions of the resin and oils need be observed in preparing the compound except that the

Mr. Kummer. quantity of oil or oils should be sufficiently in excess of the resin to facilitate its thorough solution." There is no statement anywhere giving the absolute amount of creosote used. This may have been reduced so low as to make the preservative effect nil. According to the requirements of the patent, none at all need be used, for the resin oil alone would be sufficient to facilitate the thorough solution of the resin. As a matter of fact, the writer can discover no reason for using resin oil at all except as a substitute for creosote oil. The latter will dissolve resin very effectually, and is a far more powerful antiseptic; consequently the resin oil can add neither to the solvent power of the mixture nor to its preservative qualities. Excessive use of this resin oil may be one explanation of the failure of the process.

Lacking the details of the treatment and experiments actually made, it is impossible to say whether the process was ever properly tested. It is well known that there are on record many cases of the total failure of the creosoting process. These, on careful examination, have been proven to be due entirely to improper application of the process. No one at the present time would throw discredit upon creosote as a preservative on this account.

Mr. Putnam, in his patent specification, does not give any method of applying his mixture. He states: "The compound may be applied to the timber in any suitable or well-known way." The supposition that the wood was saturated to the center may be excluded as not being practicable, consequently, the question of sterilization becomes important. There is no record, either in the patent papers or in the description of the experiments, to show that the wood was thoroughly sterilized, or even that this was taken into consideration. In fact, Mr. Putnam's own statement of the object of his process, *i. e.*, to make the wood more durable and better able to resist the attacks of insects and marine animals and the deteriorating influences of air and moisture, would lead one to suppose that he ignored entirely the great danger of germs of decay existing in the interior of the wood itself.

Finally, the writer will again call attention to the fact that Mr. Putnam's mixture would leave the timber with a very sticky surface, making it almost impossible to handle in bulk. Any or all of these causes may have contributed to his final abandonment of the process.

The statements contained in Mr. Valentine's discussion relative to paving blocks laid in Galveston are very encouraging. It would be a decided mistake to suppose that the claims made for the creo-resinate process conflict with these in the least. On the contrary, they are in perfect accordance with the prevailing opinion that creosote is an excellent preservative—the best known at the present time for this class of work. The creo-resinate process is put forward as an improvement on ordinary creosoting, yielding a product which it is claimed has cer-

tain advantageous physical properties, such as greater resistance to crushing, more elasticity, and waterproof qualities not obtainable with creosote alone, and at the same time retaining its power as a preservative. It is far from the writer's desire to throw any discredit whatever upon creosote as an antiseptic. Mr. Kummer.

With regard to the treatment of piles, discussed by Mr. Valentine, no claims have been made that the creosote-resin mixture will resist the teredo. Whether it will or will not do so is an open question until proper experiments can be made to decide this point.

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No. 884.

ON THE FLOW OF WATER OVER DAMS.

By George W. Rafter, M. Am. Soc. C. E.

Presented April 18th, 1900.

WITH DISCUSSION.

The classical experiments of the late James B. Francis, M. Am. Soc. C. E., on the flow of water over sharp-crested weirs, extended our knowledge of the general problem of weir flow considerably; and although Mr. Francis pointed out the fact that flow over sharp-crested weirs followed quite different laws from those of flow over broad and sloping crests, nevertheless it is probably true that 95% of all computations of flow over dams, made in the United States in the last twenty-five years—whatever the form of crest—have been based upon Mr. Francis' formula for sharp-crested weirs. So far has this erroneous practice proceeded that engineers have even used Mr. Francis' sharp-crested weir formula for computing flow over irregular profiles, because, in cases of litigation, Courts would accept the results without question. This is the more extraordinary because Mr. Francis himself showed, by his study of the Merrimac Dam, the considerable variation in flow resulting from change of form of crest.

Probably the main reason engineers have gone so far astray on this question has been the nearly entire lack of data applying to various

forms of crests. It is true that a few advanced hydraulicians have discussed the effect of variations in width of crest, but it has remained for H. Bazin, *Inspecteur Général des Ponts et Chaussées*, to elucidate the whole problem in a series of masterly studies, which may be found in *Annales des Ponts et Chaussées* for the years 1888, 1890, 1891, 1894, 1896 and 1898. These studies are, as regards detail and minute research, unparalleled.

Messrs. Fteley and Stearns,* have indeed shown the effect of width of crest on discharge over weirs. Their experiments, while decisive for the cases studied, are still quite limited in scope, but Bazin has determined coefficients for a very large number of cases, not only of crests of different widths, but with varying front and rear slopes, as well as for curved profiles. Indeed, taking into account the backward state of knowledge of flow over weirs, his work is in many respects revolutionary. Certainly, with the data now available, there is no excuse for using a sharp-crested weir formula in computations of flow over various-shaped crests, irregular or otherwise.

In making the foregoing comments, the writer has no intention of criticising the work of others. Indeed, he has himself groped blindly in the dark in this matter, as other engineers have done. His intention is to present an irrational practice so saliently that—with the data now at hand—from this time on, the use of sharp-crested weir formulas for computing flow over weirs of all sorts of shapes will be discontinued.

An illustration from the writer's experience is pertinent to the discussion. During the last twenty years he has had occasion to gauge streams extensively in various parts of the United States. Long experience convinced him several years ago that sharp-crested weir formulas were not applicable to dams with sloping crests, and accordingly, in arranging for an extensive series of gaugings of the Genesee River, over a sloping-faced dam at Mount Morris, New York, in 1893, he used the formula

$$Q = 3.4\, L\, H^{\frac{3}{2}}\dagger$$

as originally proposed for this dam by Augustus S. Kibbe, Jun. Am. Soc. C. E., in his report on Genesee River Storage, to John Bogart, M. Am. Soc. C. E., formerly State Engineer and Surveyor.

In 1896, after the Genesee River gaugings had been in progress at

* "Experiments on the Flow of Water, made during the Construction of Works for Conveying the Water of Sudbury River to Boston." By A. Fteley and F. P. Stearns, *Transactions*, Am. Soc. C. E., Vol. xii, p. 1.

†See Appendix F to Annual Report, State Engineer and Surveyor, for 1890, p. 455.

the dam of the Mount Morris Hydraulic Power Company for three years, a sharp-crested weir was erected on the Genesee River about 2½ miles above, where rock bottom clear across the river afforded an opportunity for such construction without heavy expense.*

TABLE No. 1.

H = head on weir, in feet.	h = head on dam, in feet.	Computed discharge over weir, in cubic feet per second, for heads = H.	Computed discharge over dam, in cubic feet per second, for heads = h.	Percentage differences in discharges.
(1)	(2)	(3)	(4)	(5)
0.50	0.60	185	135	−37.0
0.70	0.83	310	330	+ 6.0
0.80	0.90	450	445	− 1.0
1.02	1.00	540	505	− 7.0
1.86	1.55	1 325	1 260	− 5.0
2.01	1.75	1 490	1 605	+10.0
2.42	2.00	1 965	2 100	+ 7.0
2.65	2.50	2 250	3 230	+44.0
3.20	2.75	2 990	3 860	+29.0
3.78	3.00	3 840	4 554	+19.0
4.37	3.25	4 770	5 280	+11.0
4.65†	3.35†	5 240	5 590	+ 7.0

† Approximate; taken from curve.

In order to correlate the measurements at the Hydraulic Power Company's dam with those at the weir, observations were taken at each place as nearly cotemporaneously as they could be made by a man going immediately from one to the other. Table No. 1 gives some of the heads actually observed at the weir and dam, together with the discharge over the weir in comparison with the computed discharge over the dam, and the percentage differences.

The crest of the Mount Morris Dam was quite irregular, and, in order to apply weir formulas, an accurate profile was taken and the crest sub-divided into a number of approximately level sections with each section computed separately, advancing by 0.1 ft. up to 10 ft. The flow over the entire dam was obtained by adding together the sums of the several sections at the corresponding heights, and tabulating them. The zero of the gauge was at the level of the lowest section.

The computed discharges, as shown by Columns (3) and (4), are somewhat irregular. This result is due to the disturbing effect of the irregular sections of the crest, the highest point of which was 2 ft. above the lowest.

* For detailed description of this weir see Appendix VII to Annual Report, State Engineer and Surveyor, for 1896, pp. 715-19.

Column (5) shows the percentage variations between the discharges as determined by a sharp-crested weir, up to 5 200 cu. ft. per second, and the discharges computed by the formula cited. These data show at once an error in judgment, excusable only because, previous to the publication of Bazin's paper of 1898, nobody knew how to do better.

In computing the Hudson River gaugings in 1895, the writer used the formula of General Mullins, as fairly applicable to a broad-crested dam like that at Mechanicsville, where Hudson River gaugings have been kept continuously from October, 1887.*

In August, 1898, the writer began an extensive special investigation as to water supply for summit-level canals in the State of New York, for the United States Board of Engineers on Deep Waterways. The magnitude of the commercial interests involved justified a most thorough study, and the work was accordingly carried out on an extended scale. A large collection of new data has been obtained, which, by permission of the Board, the writer has the pleasure of presenting to the Society herein.

At the beginning of the study it was deemed advisable to gauge a number of streams tributary to proposed deep waterways in Central and Eastern New York, not only with reference to extending information as to the low-water flow of the Oswego, Mohawk, Black and Hudson Rivers, but especially to gain more definite information as to the flood flows of these streams and their tributaries, it being recognized clearly that the control of floods in canalized river-beds was a serious feature of the general problem.

To accomplish this, gauging stations were established on Seneca River, at Baldwinsville; Oswego River, at Fulton; Chittenango Creek, at Bridgeport; Oneida Creek, at Kenwood; West Branch of Fish Creek, at McConnellsville; East Branch of Fish Creek, above Point Rock; Salmon River, above High Falls; Mohawk River, at Ridge Mills, Little Falls and Rexford Flats; Nine Mile Creek, below Stittville; Oriskany Creek, at Oriskany and Coleman; Saquoit Creek, at New York Mills; West Canada Creek, at Dolgeville; Garoga Creek, 3 miles above mouth; Cayadutta Creek, below Johnstown, and Schoharie Creek, at Fort Hunter. In addition, gaugings of Hudson River, at Mechanicsville and Fort Edward, and of Schroon River, at Warrensburg,

*For this formula see Mullins' Irrigation Manual, pp. 11-12, 138-139, 171-172. Also see Annual Report, State Engineer and Surveyor, of New York, for 1895, pp. 105-106; and Water Supply and Irrigation Papers of United States Geological Survey, No. 24; Water Resources the of State of New York, Part I, pp. 79-80.

at stations established previously by the writer, were available, as well as of Black River, at Huntingtonville, a suburb of Watertown, at a station established by the Board of Water Commissioners of Watertown.

The foregoing gauging stations are in every case existing dams, either of masonry or timber. Several of them, as at Baldwinsville, Fulton, Little Falls, Middleville, Dolgeville, etc., have extensive power devolopments, with large quantities of water passing through turbine water wheels, for either the whole or a portion of each day. Hardly any two cross-sections are alike, as may be sufficiently appreciated by examining Figs. 8 to 20, although some of them conform generally to certain of Bazin's types, as is shown by the illustrations. Finally, many of them have gross irregularities in the crests, longitudinally, as shown. The method of treatment, in order to obtain approximately correct results, becomes, therefore, a matter of some difficulty. In a few cases, as on Nine Mile Creek, West Canada Creek, etc., where the crests were very irregular, a small amount of work was done in the way of leveling them. Generally, however, the crests were left nearly in the same condition as found, a profile was carefully taken and the crest divided into a series of approximately level sections for computation. A gauging blank was furnished the gauge readers, with columns for entering depth on crest of dam, A. M. and P. M., number of water wheels used, size of same, name of manufacturer and daily run, working head on wheels, readings of head-race and tail-race gauges, and other information necessary for keeping an accurate account of the water passing over the crest in 24 hours, as well as through water wheels for the same period. Gauge readers were employed to take these readings twice each day.

In order to obtain flows through water wheels, recourse was had to records of the test flume of the Holyoke Water Power Company, of Holyoke, Mass., where the principal wheels now in common use in New York State have, at one time or another, been tested. On requesting a record of such tests, as applying to wheels at the several gauging stations, the Holyoke Water Power Company kindly responded that they would furnish the records under the condition that they be not published unless the consent of parties for whom the wheels had been tested were first obtained. This condition being assented to, information was furnished as to tests of the principal wheels in use, giving proportional part of opening of speed gate for various conditions of tests, revolutions

of wheel, quantity of water discharged, power developed, efficiency, etc. From these records, wheel-discharge curves have been prepared for the water wheels in use at each dam. By the use of such curves, derived from actual tests, it is believed that the discharges through turbine water wheels at the various gauging stations have been computed with a very high degree of accuracy. Under these conditions turbine water wheels become in effect efficient water meters. In a few cases, where there were no tests applying, the discharges as per manufacturers' tables have been used. The writer's thanks are due to the Holyoke Water Power Company for the courtesy of furnishing these useful data.

Before describing the method of procedure for obtaining flows over dams at the several gauging stations, we may refer briefly to some of the more salient points of Bazin's papers in *Annales des Ponts et Chaussées.*

In the beginning of his first paper, Bazin remarks that the theory of the weir is the least advanced of all branches of hydraulics. The coefficients used in practice vary between such wide limits that in most cases we are unable to make a rational selection from the many numerical values assigned to them.

The problem, he says, is in fact a complicated one, being connected on the one hand with the theory of flow through orifices and on the other with that of open channels. The value of the coefficients in each case is influenced by many elements. Thus we ought to consider:

(1) The velocity of approach; that is, the velocity with which the up-stream water reaches the weir, the effect of which cannot be neglected in weirs of small height.

(2) The contraction of the vertical section of the stream at the weir, the amount depending upon the height of the weir and the form of the crest.

(3) The lateral contraction which, though unimportant in weirs of great length, seriously modifies the results in shorter weirs.

As a further condition, Bazin points out that when the down-stream channel has a width of the length of the weir, so that the overflowing sheet of water, or nappe, touches at the sides, thus preventing free admission of air under the nappe, there occur special phenomena greatly affecting the flow.*

* Bazin's earlier papers are directed specially to a detailed investigation of these several points. Space will not be taken here to describe his experiments in detail. The original data may be found in the *Annales des Ponts et Chaussées* for the years already cited. A translation of the earlier numbers has also been made by Messrs. Arthur Marichal and John C. Trautwine, Jr., and may be found in the *Proceedings* of the Engineers' Club of Philadelphia for January, 1890; July, 1892; October, 1892, and April, 1893.

Bazin's method of experimentation may be referred to briefly. A standard weir was set up at the head of a long chamber, in which the actual volume passing over was measured a sufficient number of times to give averages, which Bazin considers are accurate to within probably less than 1 per cent. Having established in this way the values of the coefficients for a standard weir, with heads varying from about 0.164 ft. to 1.969 ft., the experiments on weirs of irregular profiles were made by placing each experimental weir below the standard weir, and observing the heads synchronously on each. In these experiments a steady current was established in the channel, and observations of the known volume passing over the standard weir were made, which volume also passed over the weir under investigation, lower down.

If we let H and h denote, respectively, the head upon the standard weir and upon the lower weir, L and l, their corresponding lengths, and M and m, the coefficients of discharge, and then, adopting provisionally Formula (1) for the standard weir—

$$Q = M L H \sqrt{2gH}; \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

and similarly for the lower weir

$$Q = m l h \sqrt{2gh}. \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

Equating these two values of Q, we have

$$M L H \sqrt{H} \sqrt{2g} = m l h \sqrt{h} \sqrt{2g}, \text{ or}$$

$$M L H^{\frac{3}{2}} = m l h^{\frac{3}{2}}$$

from which we deduce the value of m:

$$m = M \left(\frac{L}{l}\right) \times \left(\frac{H}{h}\right)^{\frac{3}{2}}$$

or, conversely: $$M = m \left(\frac{l}{L}\right) \times \left(\frac{h}{H}\right)^{\frac{3}{2}}$$

As already stated, Bazin's preliminary gauging operations gave, once for all, the coefficient M for the standard weir for each value of H. The ratio $\frac{L}{l}$, which is very nearly unity, remained constant for all experiments of any one series, and, therefore, we have only to measure the heads H and h in order to obtain the coefficient m.

Fteley and Stearns experimented somewhat on the influence of the height of the weir upon the flow, and probably as interesting a point as any brought out by Bazin's extended discussion is the considerable influence of this element upon the flow. After presenting the detail

TABLE No. 2.—Values of the Coefficient m in the Formula $Q = m\,l\,h\,\sqrt{2\,g\,h}$, for a Sharp-Crested Weir without Lateral Contraction, the Air being Admitted Freely Beneath the Overflowing Sheet of Nappe.

Observed head $= h$, in feet.	Values of the coefficient m corresponding to the height p of the weir above the bottom of the channel.									Limiting value of $m = $ coefficient a.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
Value of p, in feet.	0.656	0.984	1.312	1.640	1.968	2.624	3.280	4.920	6.560	∞
0.164	0.458	0.453	0.451	0.450	0.449	0.449	0.449	0.448	0.448	0.4481
0.197	0.456	0.450	0.447	0.445	0.445	0.444	0.443	0.443	0.443	0.4427
0.230	0.455	0.448	0.445	0.443	0.442	0.441	0.440	0.440	0.439	0.4391
0.262	0.456	0.447	0.443	0.441	0.440	0.438	0.438	0.437	0.437	0.4363
0.295	0.457	0.447	0.442	0.440	0.438	0.436	0.436	0.435	0.434	0.4340
Means..	0.456	0.449	0.446	0.444	0.443	0.442	0.441	0.441	0.440	0.4400
$m\sqrt{2g}$	3.66	3.60	3.58	3.56	3.55	3.54	3.54	3.53	3.53	3.53
0.328	0.459	0.447	0.442	0.439	0.437	0.435	0.434	0.433	0.433	0.4322
0.394	0.462	0.448	0.442	0.438	0.436	0.433	0.432	0.430	0.430	0.4291
0.459	0.466	0.450	0.443	0.438	0.435	0.432	0.430	0.428	0.428	0.4267
0.525	0.471	0.453	0.444	0.438	0.435	0.431	0.429	0.427	0.426	0.4246
0.591	0.475	0.456	0.445	0.439	0.435	0.431	0.428	0.426	0.425	0.4229
Means..	0.467	0.451	0.443	0.438	0.436	0.432	0.431	0.429	0.428	0.4271
$m\sqrt{2g}$	3.74	3.62	3.56	3.52	3.50	3.47	3.46	3.44	3.44	3.43
0.656	0.480	0.459	0.447	0.440	0.436	0.431	0.428	0.425	0.423	0.4215
0.722	0.484	0.462	0.449	0.442	0.437	0.431	0.428	0.424	0.423	0.4203
0.787	0.488	0.465	0.452	0.444	0.438	0.432	0.428	0.424	0.422	0.4194
0.853	0.492	0.468	0.455	0.446	0.440	0.432	0.429	0.424	0.422	0.4187
0.919	0.496	0.472	0.457	0.448	0.441	0.433	0.429	0.424	0.422	0.4181
Means..	0.488	0.465	0.452	0.444	0.438	0.432	0.428	0.424	0.422	0.4196
$m\sqrt{2g}$	3.92	3.73	3.63	3.56	3.52	3.46	3.44	3.40	3.39	3.37
0.984	0.500	0.475	0.460	0.450	0.443	0.434	0.430	0.424	0.421	0.4147
1.050	0.500	0.478	0.462	0.452	0.444	0.436	0.430	0.424	0.421	0.4168
1.116	0.500	0.481	0.464	0.454	0.446	0.437	0.431	0.424	0.421	0.4162
1.181	0.500	0.483	0.467	0.456	0.448	0.438	0.432	0.424	0.421	0.4156
1.247	0.500	0.486	0.469	0.458	0.449	0.439	0.432	0.424	0.421	0.4150
Means..	0.500	0.481	0.464	0.454	0.446	0.437	0.431	0.424	0.421	0.4162
$m\sqrt{2g}$	4.01	3.86	3.73	3.64	3.58	3.50	3.46	3.40	3.38	3.34
1.312	0.500	0.489	0.472	0.459	0.451	0.440	0.433	0.424	0.421	0.4144
1.378	0.500	0.491	0.474	0.461	0.452	0.441	0.434	0.425	0.421	0.4139
1.444	0.500	0.494	0.476	0.463	0.454	0.442	0.435	0.425	0.421	0.4134
1.509	0.500	0.496	0.478	0.465	0.456	0.443	0.435	0.425	0.421	0.4128
1.575	0.500	0.496	0.480	0.467	0.457	0.444	0.436	0.425	0.421	0.4122
Means..	0.500	0.493	0.476	0.463	0.454	0.442	0.435	0.425	0.421	0.4133
$m\sqrt{2g}$	4.01	3.96	3.82	3.72	3.64	3.55	3.49	3.41	3.38	3.32
1.640	0.500	0.496	0.482	0.468	0.459	0.445	0.437	0.426	0.421	0.4118
1.706	0.500	0.496	0.483	0.470	0.460	0.446	0.438	0.426	0.421	0.4112
1.772	0.500	0.496	0.485	0.472	0.461	0.447	0.438	0.426	0.421	0.4107
1.837	0.500	0.496	0.487	0.473	0.463	0.448	0.439	0.427	0.421	0.4101
1.903	0.500	0.496	0.489	0.475	0.464	0.449	0.440	0.427	0.421	0.4096
1.969	0.500	0.496	0.490	0.476	0.466	0.451	0.441	0.427	0.421	0.4092
Means..	0.500	0.496	0.486	0.472	0.462	0.448	0.436	0.427	0.421	0.4104
$m\sqrt{2g}$	4.01	3.98	3.90	3.79	3.71	3.60	3.50	3.42	3.38	3.29

of experiments on sharp-crested weirs of various heights and for various heads between the limits stated, Bazin gives a table of values of the coefficient m for sharp-crested weirs, ranging in height from 0.656 ft. to 6.56 ft. (0.2 to 2.0 m.). Column (11) of Table No. 2 gives the limiting value of m, which equals the coefficient μ of Bazin's formulas, (represented by n) and which represents the value of m for a weir of infinite height, or of such a height that the height of the weir above the bottom of the channel has no further effect upon the flow. As will be seen by examining this table, the influence beyond 6.56 ft. is only slight.

The formula for a sharp-crested weir without end contractions, as ordinarily used in the United States, is that of Mr. Francis, namely:

$$Q = Cl H^{\frac{3}{2}}.$$

In this formula, it is necessary to take into account the velocity of approach. In the formula as it here stands, we may consider h as the uncorrected form, while we may consider that in H all corrections are made. Usually, in practice, such corrections are conveniently made by adding the correction factor to the formula, a simple form of tabulation being used.

In Bazin's formula it is equally necessary to take into account the velocity of approach, although his coefficient, m, apparently includes that element within itself. This is, however, for convenience merely, the more especially because within the range of his experiments the velocity of approach was mostly a relatively unimportant element. Nevertheless, Bazin gives elaborate corrections for its use, when necessary. It is, however, very important to understand that, while Bazin's experiments do not include a separate correction for velocity of approach, but that its effect is included in m, nevertheless, when, as in the present case, very high heads are involved, the correction can be more conveniently made by a separate correction factor than in any other manner, and accordingly this is the method which has been applied in the Cornell University experiments following. These coefficients are, therefore, corrected for velocity of approach, and are ready to be used without further corrections of any kind. The values of $m\sqrt{2g}$ for different heads on the crest and for heights of weir varying from 0.656 ft. to 6.56 ft., and also for the limiting value of $m = n$ may be seen as carried out in Table No. 2.

In regard to the accuracy of the coefficients given by this table, Bazin remarks that, except in the unusual case of a very low weir, which should always be avoided, it will give the coefficient m within 1%, provided, however, that the arrangements of his standard weir be exactly reproduced. It is also pointed out as especially important that the admission of air behind the falling sheet be perfectly assured; otherwise m may vary within much wider limits.

Description of Bazin's Experiments on Weirs of Irregular Cross-Section.

The following statements have been condensed from Bazin's paper in *Annales des Ponts et Chaussées*, for 1898.

We will now consider weirs in which the back and front faces, instead of being vertical, have a slope of greater or less inclination. The conditions of discharge will be found to be greatly modified. The slope on the up-stream side tends to diminish the contraction in passing over the crest, and hence to increase the discharge. The influence of the down-stream slope, on the other hand, is not always constant, but varies according to the degree of inclination. If the down-stream face is not far from vertical, the nappe adheres to it for small discharges, but becomes detached at a certain head and then follows the condition described as wetted underneath, analogous to that which we have studied for square timber weirs. If, on the contrary, the slope is nearly horizontal, the nappe does not detach itself, but remains in contact with the face of the weir under all heads. The discharge may, however, vary greatly according to the degree of inclination of the face. On the other hand, the influence of the width of crest is considerable, as we have seen in the case of weirs formed of square timbers. Hence, a weir with a wide crest and inclined faces may present a large variety of results, each type having, so to speak, its own special scale of coefficients. Such a study, to be complete, must include a very considerable number of particular cases. Without embracing all possible cases, the experiments made have been somewhat numerous. They include weirs with different degrees of slope on the front and back faces, and having sharp crests, on the one hand, and on the other, crests 0.10, 0.20 and 0.40 m. in width (0.328, 0.656 and 1.312 ft., respectively).

Some additional experiments have been made on weirs having crests joined to the inclined faces by curved surfaces, and, finally, weirs having completely curved profiles have been experimented upon.

We may divide the numerous series of experiments into five groups, namely:

(1) Weirs so nearly vertical on the down-stream side that the nappe remains detached.

(2) Weirs having the up-stream face vertical, or nearly vertical, but with a slope on the down-stream face so nearly horizontal that the water always remains in contact.

(3) Weirs both faces of which are at an inclination differing from the horizontal by less than 45 degrees.

(4) Weirs in which the crest is joined to the inclined faces by curved surfaces.

(5) Weirs having completely curved profiles.

First Group.—Weirs Having the Down-Stream Face Vertical or Nearly Vertical.

This group includes ten series of experiments, the results of which differ notably, according to inclination of slope and width of crest. The values of the coefficient $\frac{m}{m'}$,* which have been obtained for sharp-crested weirs, have been compared with those corresponding to nappes wetted underneath with sharp-crested weirs. Similarly, we may compare conveniently the values obtained for weirs with crests 0.10 and 0.20 m. wide (0.328 and 0.656 ft., respectively), with those for flat-crested beam weirs of the same width in the table.†

The coefficients have been made to follow a uniform law, by plotting the immediate results of the experiments in such a manner that the head h represents the abscissa and the ratio $\frac{m}{m'}$ the ordinate of a point representing one of the experiments. A mean curve has been drawn by the aid of these points, and from this curve, drawn on a large scale, values of the ratio $\frac{m}{m'}$ have been taken, corresponding, roundly, to abscissas of 0.10 m., 0.15 m., etc. (0.328 ft., 0.492 ft., etc.). All the ex-

* This may be defined as the ratio of the coefficient of the weir under special consideration to that of the standard weir of comparison.

† See p. 159, *Annales des Ponts et Chaussées, 2d Trimestre*, 1898.

periments terminate with nappes wetted underneath, but the discharge for these, as well as for depressed and adhering nappes, is shown in the table, the nature of the nappe in each case being indicated in the proper column. If we consider, first, sharp-crested weirs, we perceive that the appearance of the wetted nappe is preceded by the depressed nappe, imprisoning air between its under surface and the body of the weir, excepting in the case of a weir with a face batter of 3 : 2, which permits the formation of an adhering nappe. When once the wetted nappe is established, its coefficient does not differ greatly from that for a sharp-crested weir, excepting in the cases where the slope on the back is 3 : 1 and 3 : 2, when it is greater. For weirs with flat crests 0.10 m. (0.328 ft.) wide, the adhering form of nappe appears when the up-stream face has a slope of 3 : 1 or 3 : 2. The ratio $\frac{m}{m'}$ is also modified, however, by adherence of the water to the flat crest. At the moment when this adhesion ceases, the coefficient diminishes suddenly about 10 per cent. This has taken place in the two series, Nos. 133 and 134, where the up-stream face is vertical. In the other series, the head has not been sufficiently large to detach the nappe, and the coefficient remains, in these experiments, greater in value than it would be for a beam weir in which the nappe has become detached before the given head is attained.

Only one series of experiments has been made on weirs with crest 0.20 m. (0.656 ft.) wide, the down-stream face being vertical and that on the up-stream side having a slope of 1 : 2. In accordance with the increased width of crest the wetted nappe does not appear until quite late, and with a coefficient notably diminished. It is clear that more extended experiments would have led in all cases to results differing in a similar manner from those for crests 0.10 m. in width. Without going farther we may say that, on a weir having the down-stream face nearly vertical, the wetted form of nappe always appears when a certain head has been attained. This limiting head varies with the inclination of the front and back faces, and is never the same as that which corresponds to detachment of the nappe from the flat crest, which also influences the value of the coefficient. Each type of weir requires a special study, and, in view of the complexity of conditions involved, it is impossible to establish a general formula for the discharge coefficient.

Second Group.—Face of the Weir on the Up-Stream Side Vertical or Nearly Vertical.

Let us now consider the case where, in contradistinction to the first group, the back is nearly vertical and the down-stream face has considerable slope. For sharp-crested weirs with the up-stream face vertical the ratio $\frac{m}{m'}$ is nearly constant for each series, the mean values being 1.13, 1.03, 0.90 and 0.84 for slopes on the down-stream face of 1 : 1, 1 : 2, 1 : 5 and 1 : 10, respectively. Where the up-stream face has a slope of 3 : 1 or 2 : 1, it produces an increase, in the value of the coefficient, of a few per cent. Turning to the experiments on weirs with crests 0.10, 0.20 and 0.40 m. (0.328, 0.656 and 1.312 ft., respectively) in width, it may be seen that the coefficients increase with the head, Series No. 143 only showing, for the final values, a rapid diminution, proving that the nappe has become detached. It is necessary, in fact, in order that the nappe shall become detached, that the head shall be greater in proportion as the width of the crest becomes greater and the slope of the down-stream face more gentle. This limit has not, in general, been reached in the experiments, and the nappes remain attached to the crests, excepting for Series No. 143 (slope on the down-stream side 1 : 1), where the detachment of the nappe leads to a diminution of the coefficient ratio from 1.21 to 1.14. The next series, No. 144, indicates also a slight tendency of the ratio $\frac{m}{m'}$ to decrease, the slope here being 1 : 2. This tendency entirely disappears where the slope of the down-stream face is not greater than 1 : 3 or 1 : 4. Omitting the results for the relatively small heads of 0.10 m. (0.328 ft.) or less, where there are some irregularities, the increments of the coefficient ratio $\frac{m}{m'}$ for heads from 0.10 m. (0.328 ft.) to the limits of the experiments, are given for each series, in comparison with the results for a weir with a crest 0.10 m. (0.328 ft.) in width, Series Nos. 133 and 134, and with those for flat-crested weirs, 0.40 m. (1.312 ft.) and 2.00 m. (6.56 ft.) in width, Series Nos. 113 and 115. Such comparison shows, first, that for the same width of crest, $\frac{m}{m'}$ diminishes when the inclination of the down-stream face is gradually diminished below 45°; second, that, other things being equal, that is to say, for the same slope on the two faces in each case, the ratio $\frac{m}{m'}$ diminishes when the width of crest is increased.

Third Group.—Slope of the Up-Stream and Down-Stream Faces Very Gentle, not Exceeding 45 Degrees.

Weirs encountered in practice do not often have nearly vertical faces like those we have been considering, but have slopes inclined 45°, or more, from the vertical. Such weirs have been made the subject of a series of experiments, in which slopes of 1 : 1 and 1 : 2 on the up-stream side have been combined with slopes of 1 : 1, 1 : 2 and 1 : 5 on the down-stream side for three different widths of crest.

In most cases the ratio $\frac{m}{m'}$ increases with the head, but, in order to investigate more fully the changes in value of $\frac{m}{m'}$, it is necessary to consider separately the case of sharp-crested weirs as distinguished from those with wide crests.

Sharp-Crested Weirs.

Slope 1 : 1 on Down-Stream Side.—The coefficient ratio $\frac{m}{m'}$ decreases as the head h increases from above 1.20 for very slight heads to 1.11 or 1.12 for the greatest heads used. Its value is sensibly the same for the two slopes of 1 : 1 and 1 : 2 on the back. The rate of decrease is not uniform, being very gradual to $h = 0.30$ m. (0.984 ft.), beyond which it changes rapidly, without doubt due to the detachment of the nappe.

Slope 1 : 2 on Down-Stream Side.—Instead of decreasing as the head increases, $\frac{m}{m'}$ increases slowly in value between the limits 1.10 and 1.13, its value being nearly the same for the two slopes of 1 : 1 and 1 : 2 on the back.

Slope 1 : 5 on Down-Stream Side.—In this case the coefficient ratio is nearly independent of h, decreasing from 1.015 to 1.000 for a slope of 1 : 1 on the back, and from 1.045 to 1.035 for a slope of 1 : 2 on the back.

Weirs Having Crests 0.10 and 0.20 M. (0.328 and 0.656 Ft.) in Width.

The coefficient always increases with the head, but the limits between which this increase takes place differ in each case. If it were possible to increase the head indefinitely, and at the same time the height of the weir, the conditions of discharge would approach progressively those for a sharp-crested weir, the width of the crest

becoming more and more negligible, relative to the general dimensions of the dam. The series of coefficients relating to crests of a given width cannot be, with certainty, extended beyond the experimental limits between which they were obtained. If, however, the curves representing the coefficients were prolonged sufficiently, they would converge toward those which correspond to a sharp-crested weir with vertical faces. The slope on the back determines the direction of the filaments which constitute the inferior surface of the nappe, and influences thus the contraction at the inner edge of the crest, and hence also the discharge. The slope on the down-stream side, on the other hand, affects the pressure underneath the nappe.

When width of crest is not negligible, the inclination of the down-stream face determines the limiting head at which the nappe detaches itself from the crest, and at which point the conditions of discharge change suddenly. This limit depends also, in a certain measure, on the velocity of approach, or, which amounts to the same thing, on the ratio of the head h to the height p of the weir. A complete formula should include, in addition to the slopes of the two faces, the two ratios, $\frac{h}{c}$ and $\frac{h}{p}$, where c is the width of the flat portion of the crest. Such a formula would be excessively complicated.

Fourth Group.—The Two Faces United by a Curved Surface at the Crest of the Weir.

Seven types of weirs were experimented on, with a view to determining the effect on the discharge of joining the two faces of the weir by a curved surface, the slopes used being, on the up-stream side, very nearly vertical ($\frac{1}{6}$: 1), and on the down-stream side from 3 : 1 to 5 : 1. The up-stream edge of the crest being rounded to an arc of 0.05, 0.10 or 0.20 m. (0.164, 0.328 or 0.656 ft.) radius.

Types Nos. 1 and 2 differ only in regard to the radius of curvature of the back edge of the crest, this being 0.05 m. (0.164 ft.) for the first, and 0.10 m. (0.328 ft.) for the second. The radius of 0.10 m. increases the discharge slightly more than that of 0.05 m., though the difference is unimportant, but the values of $\frac{m}{m'}$ in both cases surpass considerably those which have been obtained for similar weirs with the two

faces united by flat crests 0.10 and 0.20 m. (0.328 and 0.656 ft.) in width. Series Nos. 145 and 153.

Types Nos. 3 and 4 are similar to Types Nos. 1 and 2, except as regards the inclination on the down-stream side, which has been reduced from 1 : 3 to 1 : 5. This conduces to equalize the values of m, which differ relatively little from each other, although the values are sensibly less at the same heads than those for Types Nos. 1 and 2, and yet somewhat greater than for weirs with flat crests, 0.10 and 0.20 m. (0.328 and 0.656 ft.) in width. Series Nos. 145, 155 and 156.

Types Nos. 5 and 6 differ only by the radius of curvature at the back, which is 0.10 and 0.20 m., respectively, the crest being wider than in the preceding cases. The value of m is somewhat less than before, not differing greatly from that which corresponds to Types Nos. 3 and 4.

The width of crest is increased still further in Type No. 7, the length of the rectilinear section between the origins of the two curved surfaces being 0.20 m. (0.656 ft.). This modification produces a sensible diminution in the value of m.

Fifth Group.—Weirs with Completely Curved Profiles.

We turn, finally, to the consideration of weirs having completely curved profiles. The coefficient m then attains exceptionally high values. Types Nos. 1 and 2, having vertical down-stream faces, permit of the formation of nappes wetted underneath. The corresponding coefficients are much higher than for the analogous case with a sharp-crested weir.*

Types Nos. 3 and 4 have crests which differ only in the radii of curvature of the curved surfaces, being 0.05 and 0.08 m. (0.164 and 0.2624 ft.), respectively, for Type No. 3, and 0.10 and 0.12 m. (0.328 and 0.3936 ft.), respectively, for Type No. 4. The crest, in this latter case, is much larger and the coefficient m is, as a result, notably less for heads up to 0.30 m. (0.984 ft.), but it is not the same beyond this point, for the concave form of Type No. 3 tends to produce detachment of the nappe, and the coefficient for this type continues to diminish from this point, becoming less than for Type No. 4, in which the coefficient increases for all heads within the limits of the experiments.

* For the type-forms under this group, see Bazin's paper.

The results of Bazin's experiments on weirs of irregular profiles, except a few of the series which have been omitted, will be found on the pages indicated in the following list. The value of $m\sqrt{2g}$ is given in every case as derived from Bazin's tabulated value of m:

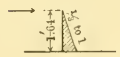
BAZIN'S EXPERIMENTS.

Series No.	Page.	Series No.	Page.	Series No.	Page.
113	237	140	241	163	248
114	275*	141	242	164	248
115	277*	142	242	165	248
116	276*	143	242	166	249
117	278*	144	243	167	249
125	237	145	243	168	249
126	237	146	243	170	250
127	238	147	244	172	250
128	238	149	244	173	274*
129	238	150	244	175	250
130	267*	151	245	176	251
131	239	153	245	177	251
132	239	154	245	178	273*
133	239	156	246	179	251
134	240	157	246	181	252
135	268*	158	246	182	252
136	240	159	247	188	252
137	240	160	247	189	253
138	241	161	247	193	253
139	241	162	272*		

* With diagram of similar Cornell experiment.

	No. of Experiment.	h=observed depth on crest, in feet.	m=coefficient of discharge.	$m\sqrt{2g}$	
BAZIN'S SERIES NO. 113	1	0.208	0.3294	2.64	
	2	0.289	0.3313	2.66	
	3	0.363	0.3307	2.66	
	4	0.443	0.3302	2.65	
	5	0.518	0.3321	2.66	
	6	0.592	0.3348	2.64	
	7	0.667	0.3378	2.71	
	8	0.736	0.3427	2.75	
	9	0.805	0.3468	2.78	
	10	0.863	0.3494	2.80	
	11	0.936	0.3547	2.85	
	12	0.989	0.3593	2.88	
	13	1.035	0.3628	2.91	
	14	1.076	0.3660	2.94	
	15	1.111	0.3682	2.95	
	16	1.159	0.3734	3.00	
	17	1.197	0.3758	3.01	
	18	1.252	0.3821	3.06	
	19	1.320	0.3877	3.11	
BAZIN'S SERIES NO. 125	1	0.318	0.4516	3.62	Nappe depressed.
	2	0.383	0.4530	3.63	
	3	0.439	0.4604	3.69	
	4	0.492	0.4715	3.79	
	5	0.532	0.4978	4.00	
	6	0.569	0.5258	4.22	Nappe wetted underneath.
	7	0.633	0.5178	4.16	
	8	0.691	0.5187	4.16	
	9	0.752	0.5100	4.09	
	10	0.818	0.5082	4.09	
	11	0.878	0.5047	4.05	
	12	0.950	0.4896	4.00	
	13	1.006	0.4996	4.01	
	14	1.077	0.4954	3.97	
	15	1.140	0.4905	3.93	
	16	1.190	0.4919	3.95	
	17	1.266	0.4892	3.92	
	18	1.328	0.4883	3.91	
	19	1.395	0.4863	3.90	
BAZIN'S SERIES NO. 126	1	0.295	0.5015	4.02	Nappe depressed.
	2	0.360	0.5028	4.03	
	3	0.413	0.5024	4.03	
	4	0.467	0.5138	4.12	
	5	0.520	0.5205	4.17	
	6	0.567	0.5332	4.27	Nappe wetted underneath.
	7	0.627	0.5256	4.21	
	8	0.684	0.5238	4.20	
	9	0.740	0.5251	4.21	
	10	0.804	0.5241	4.20	
	11	0.860	0.5234	4.20	
	12	0.920	0.5233	4.20	
	13	0.981	0.5203	4.17	
	14	1.040	0.5228	4.19	
	15	1.105	0.5175	4.15	
	16	1.164	0.5197	4.17	
	17	1.222	0.5170	4.15	
	18	1.284	0.5187	4.16	
	19	1.351	0.5142	4.12	

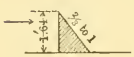
BAZIN'S SERIES NO. 127.*



No. of experiment.	h = observed depth on crest, in feet.	m = coefficient of discharge.	$m\sqrt{2g}$	
1	0.324	0.4326	3.47	Nappe depressed
2	0.389	0.4381	3.51	
3	0.449	0.4419	3.54	
4	0.509	0.4454	3.57	
5	0.561	0.4606	3.69	
6	0.608	0.4728	3.80	
7	0.653	0.4933	3.95	
8	0.697	0.5072	4.07	Nappe wetted
9	0.761	0.4989	4.00	underneath.
10	0.831	0.4935	3.95	
11	0.896	0.4895	3.93	
12	0.961	0.4885	3.92	
13	1.033	0.4812	3.86	
14	1.093	0.4823	3.87	
15	1.166	0.4720	3.79*	
16	1.228	0.4746	3.81	
17	1.292	0.4729	3.80	
18	1.360	0.4726	3.79	
19	1.421	0.4715	3.78	

*See Bazin's observations relative to the behavior of the nappe for Series Nos. 127 and 128.

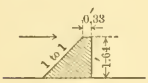
BAZIN'S SERIES NO. 128.*



1	0.295	0.5015	4.02	Nappe adhering.
2	0.353	0.5104	4.09	
3	0.407	0.5090	4.09	
4	0.464	0.5107	4.10	
5	0.526	0.5068	4.07	
6	0.580	0.5159	4.14	
7	0.626	0.5183	4.16	
8	0.692	0.5179	4.15	
9	0.737	0.5156	4.14	
10	0.805	0.5178	4.15	
11	0.843	0.5133	4.12	
12	0.890	0.4909	3.94	Nappe wetted
13	0.925	0.4955	3.97	underneath.
14	0.967	0.4849	3.84	
15	1.023	0.4834	3.87	
16	1.096	0.4820	3.87	
17	1.166	0.4766	3.83	
18	1.230	0.4758	3.82	
19	1.294	0.4725	3.79	
20	1.360	0.4733	3.80	
21	1.430	0.4696	3.77	

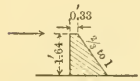
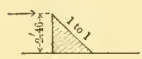
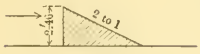
*See Bazin's observations relative to the behavior of the nappe for Series Nos. 127 and 128.

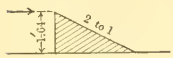
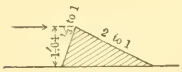
BAZIN'S SERIES NO. 129.



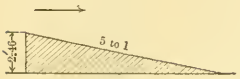
1	0.343	0.4064	3.25	Nappe depressed
2	0.399	0.4225	3.39	
3	0.456	0.4343	3.48	
4	0.511	0.4463	3.57	
5	0.565	0.4577	3.67	
6	0.612	0.4722	3.79	Nappe wetted
7	0.665	0.4778	3.83	underneath and
8	0.726	0.4838	3.88	attached to flat
9	0.774	0.4952	3.97	crest.
10	0.830	0.4982	4.00	
11	0.885	0.5029	4.04	
12	0.943	0.5079	4.08	
13	0.995	0.5116	4.11	
14	1.047	0.5155	4.14	
15	1.102	0.5187	4.16	
16	1.163	0.5174	4.15	
17	1.212	0.5236	4.21	
18	1.266	0.5263	4.23	
19	1.327	0.5276	4.23	

	No. of experiment.	h = observed depth on crest, in feet.	m = coefficient of discharge.	$m\sqrt{2g}$	
BAZIN'S SERIES NO. 131	1	0.347	0.3989	3.20	Nappe depressed.
	2	0.336	0.4140	3.31	Nappe adhering.
	3	0.395	0.4298	3.45	
	4	0.450	0.4326	3.47	
	5	0.504	0.4443	3.56	
	6	0.564	0.4611	3.70	
	7	0.610	0.4755	3.82	
	8	0.667	0.4770	3.83	
	9	0.720	0.4882	3.91	
	10	0.773	0.4937	3.96	Nappe wetted underneath and attached to flat crest.
	11	0.835	0.4948	3.97	
	12	0.889	0.4997	4.01	
	13	0.943	0.5048	4.05	
	14	0.998	0.5090	4.08	
	15	1.046	0.5128	4.12	
	16	1.098	0.5172	4.15	
	17	1.161	0.5167	4.15	
	18	1.213	0.5226	4.20	
	19	1.264	0.5249	4.21	
	20	1.333	0.5242	4.21	
	21	1.394	0.5234	4.20	
BAZIN'S SERIES NO. 132	1	0.337	0.4128	3.31	Nappe adhering.
	2	0.398	0.4351	3.41	
	3	0.453	0.4344	3.48	
	4	0.509	0.4488	3.60	
	5	0.563	0.4609	3.70	
	6	0.619	0.4663	3.74	
	7	0.665	0.4788	3.84	
	8	0.719	0.4876	3.92	
	9	0.779	0.4921	3.95	
	10	0.826	0.5029	4.03	
	11	0.874	0.5108	4.10	
	12	0.926	0.5180	4.16	
	13	0.974	0.5237	4.21	
	14	1.047	0.5136	4.12	Nappe wetted underneath and attached to flat crest.
	15	1.099	0.5174	4.15	
	16	1.165	0.5177	4.15	
	17	1.212	0.5225	4.19	
	18	1.271	0.5238	4.21	
	19	1.328	0.5260	4.22	
BAZIN'S SERIES NO. 133	1	0.349	0.3846	3.09	Nappe depressed.
	2	0.408	0.4028	3.23	
	3	0.460	0.4127	3.31	Nappe adhering.
	4	0.456	0.4266	3.42	
	5	0.506	0.4445	3.57	
	6	0.565	0.4540	3.64	
	7	0.610	0.4692	3.76	
	8	0.666	0.4775	3.83	
	9	0.707	0.4922	3.95	
	10	0.767	0.4979	4.00	
	11	0.816	0.5079	4.07	
	12	0.870	0.5122	4.11	Nappe wetted underneath and attached to flat crest.
	13	0.928	0.5151	4.13	
	14	0.978	0.5208	4.17	
	15	1.033	0.5224	4.19	
	16	1.095	0.5243	4.21	
	17	1.205	0.4889	3.92	Nappe wetted underneath but detached from flat crest.
	18	1.283	0.4794	3.84	
	19	1.276	0.4842	3.88	
	20	1.346	0.4793	3.84	
	21	1.415	0.4783	3.83	

	No. of experiment.	h = observed depth on crest, in feet.	m = coefficient of discharge.	$m\sqrt{2g}$	
BAZIN'S SERIES No. 134 	1	0.350	0.3880	3.11	Nappe adhering.
	2	0.405	0.4090	3.28	
	3	0.458	0.4228	3.39	
	4	0.513	0.4378	3.51	
	5	0.567	0.4515	3.62	
	6	0.615	0.4633	3.71	
	7	0.664	0.4751	3.81	
	8	0.714	0.4867	3.91	
	9	0.767	0.4970	3.99	
	10	0.815	0.5061	4.06	
	11	0.866	0.5186	4.16	
	12	0.909	0.5280	4.24	
	13	0.964	0.5334	4.28	
	14	1.011	0.5380	4.32	
	15	1.060	0.5427	4.36	
	16	1.118	0.5463	4.38	
	17	1.253	0.4957	3.98	Nappe wetted underneath but detached from flat crest.
	18	1.348	0.4804	3.85	
	19	1.574	0.4764	3.82	
	20	1.489	0.4712	3.78	
BAZIN'S SERIES No. 136 	1	0.183	0.4865	3.90	
	2	0.241	0.4815	3.86	
	3	0.304	0.4803	3.85	
	4	0.361	0.4814	3.86	
	5	0.421	0.4831	3.88	
	6	0.484	0.4820	3.87	
	7	0.542	0.4842	3.88	
	8	0.597	0.4854	3.89	
	9	0.658	0.4874	3.91	
	10	0.713	0.4886	3.92	
	11	0.776	0.4900	3.93	
	12	0.830	0.4948	3.97	
	13	0.887	0.4941	3.96	
	14	0.953	0.4962	3.98	
	15	1.010	0.4950	3.97	
	16	1.068	0.4979	4.00	
	17	1.122	0.4975	3.99	
	18	1.179	0.4997	4.01	
	19	1.244	0.4993	4.01	
	20	1.299	0.5005	4.01	
	21	1.361	0.5023	4.03	
BAZIN'S SERIES No. 137 	1	0.268	0.4324	3.47	
	2	0.339	0.4310	3.45	
	3	0.391	0.4359	3.50	
	4	0.451	0.4323	3.47	
	5	0.513	0.4400	3.53	
	6	0.578	0.4377	3.51	
	7	0.637	0.4378	3.51	
	8	0.700	0.4434	3.55	
	9	0.765	0.4437	3.56	
	10	0.822	0.4411	3.56	
	11	0.887	0.4445	3.56	
	12	0.946	0.4513	3.62	
	13	1.012	0.4476	3.59	
	14	1.078	0.4505	3.61	
	15	1.142	0.4490	3.60	
	16	1.201	0.4517	3.62	
	17	1.262	0.4520	3.62	
	18	1.322	0.4513	3.61	

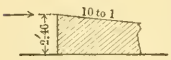
	No. of experiment.	h = observed depth on crest, in feet.	m = coefficient of discharge.	$m\sqrt{2g}$
BAZIN'S SERIES No. 138. 	1	0.194	0.4450	3.57
	2	0.263	0.4357	3.50
	3	0.327	0.4344	3.48
	4	0.391	0.4356	3.50
	5	0.447	0.4429	3.56
	6	0.510	0.4524	3.63
	7	0.571	0.4511	3.62
	8	0.626	0.4622	3.71
	9	0.685	0.4565	3.66
	10	0.745	0.4597	3.69
	11	0.807	0.4611	3.70
	12	0.873	0.4637	3.72
	13	0.927	0.4643	3.72
	14	0.992	0.4683	3.76
	15	1.045	0.4743	3.80
	16	1.110	0.4707	3.78
	17	1.176	0.4716	3.79
	18	1.233	0.4746	3.81
	19	1.289	0.4758	3.82
	20	1.355	0.4758	3.82
	21	1.427	0.4778	3.83
BAZIN'S SERIES No. 139. 	1	0.190	0.4561	3.66
	2	0.253	0.4585	3.68
	3	0.312	0.4641	3.72
	4	0.375	0.4570	3.66
	5	0.434	0.1651	3.73
	6	0.500	0.4643	3.72
	7	0.552	0.4706	3.78
	8	0.615	0.4693	3.76
	9	0.667	0.4761	3.82
	10	0.733	0.4725	3.79
	11	0.798	0.4734	3.80
	12	0.852	0.4789	3.84
	13	0.915	0.4808	3.86
	14	0.969	0.4821	3.87
	15	1.023	0.4883	3.92
	16	1.092	0.4856	3.90
	17	1.151	0.4868	3.90
	18	1.210	0.4909	3.94
	19	1.258	0.4927	3.95
	20	1.326	0.4904	3.93
	21	1.394	0.4898	3.93
BAZIN'S SERIES No. 140.	1	0.191	0.4696	3.77
	2	0.252	0.4666	3.75
	3	0.308	0.4677	3.75
	4	0.371	0.4620	3.71
	5	0.436	0.4699	3.77
	6	0.487	0.4666	3.75
	7	0.549	0.4747	3.81
	8	0.604	0.4761	3.82
	9	0.664	0.4783	3.83
	10	0.719	0.4792	3.84
	11	0.785	0.4842	3.88
	12	0.837	0.4843	3.88
	13	0.905	0.4889	3.92
	14	0.961	0.4861	3.90
	15	1.023	0.4920	3.95
	16	1.080	0.4905	3.93
	17	1.143	0.4951	3.97
	18	1.195	0.4930	3.96
	19	1.254	0.4951	3.97
	20	1.316	0.4978	3.99
	21	1.375	0.5000	4.01

BAZIN'S SERIES No. 141.



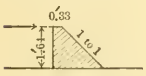
No. of experiment.	h = observed depth on crest, in feet.	m = coefficient of discharge.	$m\sqrt{2g}$	
1	0.214	0.3771	3.02	
2	0.281	0.3854	3.09	
3	0.355	0.3831	3.07	
4	0.425	0.3790	3.04	
5	0.489	0.3836	3.08	
6	0.561	0.3846	3.08	
7	0.624	0.3957	3.17	
8	0.692	0.3882	3.11	
9	0.758	0.3901	3.12	
10	0.822	0.3933	3.15	
11	0.888	0.3960	3.17	
12	0.956	0.3977	3.19	
13	1.029	0.3978	3.19	
14	1.113	0.3955	3.17	
15	1.165	0.3996	3.21	
16	1.237	0.3987	3.20	
17	1.298	0.4016	3.22	
18	1.369	0.4016	3.22	
19	1.431	0.4047	3.24	
20	1.463	0.4051	3.25	

BAZIN'S SERIES No. 142.



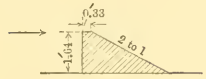
1	0.300	0.3537	2.83	
2	0.369	0.3624	2.90	
3	0.447	0.3582	2.87	
4	0.509	0.3565	2.86	
5	0.591	0.3594	2.88	
6	0.666	0.3572	2.86	
7	0.727	0.3639	2.92	
8	0.795	0.3660	2.94	
9	0.861	0.3661	2.94	
10	0.934	0.3681	2.95	
11	1.007	0.3677	2.95	
12	1.079	0.3710	2.98	
13	1.149	0.3716	2.98	
14	1.222	0.3726	2.99	
15	1.285	0.3738	3.00	
16	1.362	0.3742	3.00	
17	1.430	0.3752	3.01	

BAZIN'S SERIES No. 143.



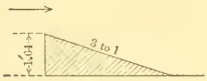
1	0.352	0.3886	3.11	Nappe adhering and
2	0.415	0.3985	3.19	attached to flat crest.
3	0.462	0.4250	3.41	
4	0.513	0.4386	3.52	
5	0.567	0.4515	3.62	
6	0.619	0.4608	3.70	
7	0.675	0.4704	3.77	
8	0.725	0.4803	3.85	
9	0.771	0.4938	3.96	
10	0.821	0.5059	4.06	
11	0.874	0.5135	4.12	
12	0.920	0.5208	4.18	
13	0.972	0.5283	4.24	
14	1.022	0.5321	4.27	
15	1.072	0.5404	4.34	
16	1.126	0.5435	4.36	
17	1.172	0.5489	4.40	
18	1.227	0.5514	4.42	
19	1.331	0.5250	4.21	Nappe adhering but
20	1.369	0.5229	4.20	detached from flat crest.

BAZIN'S SERIES NO. 144



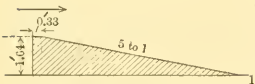
No. of experiment	h = observed depth on crest, in feet.	m = coefficient of discharge.	$m\sqrt{2g}$
1	0.358	0.3808	3.05
2	0.419	0.3954	3.17
3	0.475	0.4081	3.27
4	0.533	0.4205	3.37
5	0.592	0.4255	3.41
6	0.648	0.4372	3.50
7	0.700	0.4433	3.55
8	0.757	0.4531	3.63
9	0.810	0.4588	3.68
10	0.865	0.4679	3.75
11	0.919	0.4714	3.78
12	0.978	0.4777	3.83
13	1.034	0.4774	3.83
14	1.104	0.4759	3.82
15	1.164	0.4766	3.82
16	1.232	0.4769	3.83
17	1.286	0.4784	3.84
18	1.351	0.4782	3.84
19	1.411	0.4811	3.86

BAZIN'S SERIES NO. 145



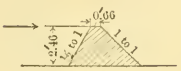
1	0.359	0.3765	3.02
2	0.424	0.3870	3.10
3	0.479	0.3971	3.18
4	0.547	0.4059	3.25
5	0.598	0.4177	3.35
6	0.658	0.4218	3.38
7	0.720	0.4262	3.42
8	0.781	0.4322	3.47
9	0.835	0.4358	3.49
10	0.902	0.4409	3.53
11	0.962	0.4409	3.53
12	1.032	0.4425	3.53
13	1.087	0.4461	3.58
14	1.152	0.4474	3.58
15	1.210	0.4505	3.61
16	1.274	0.4502	3.61
17	1.331	0.4515	3.62
18	1.396	0.4538	3.64
19	1.467	0.4544	3.64

BAZIN'S SERIES NO. 146



1	0.367	0.3632	2.91
2	0.437	0.3693	2.97
3	0.492	0.3837	3.08
4	0.567	0.3801	3.05
5	0.621	0.3923	3.14
6	0.695	0.3885	3.11
7	0.749	0.3987	3.20
8	0.817	0.3988	3.20
9	0.883	0.4021	3.22
10	0.951	0.4024	3.22
11	1.012	0.4080	3.27
12	1.085	0.4072	3.26
13	1.141	0.4119	3.30
14	1.213	0.4124	3.30
15	1.272	0.4153	3.33
16	1.344	0.4169	3.34
17	1.405	0.4190	3.36
18	1.473	0.4211	3.38
19	1.532	0.4233	3.39

BAZIN'S SERIES NO. 147

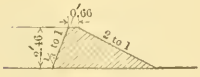


No. of experiment	h = observed depth on crest, in feet.	m = coefficient of discharge	$m\sqrt{2g}$
1	0.231	0.3429	2.75
2	0.308	0.3552	2.85
3	0.373	0.3566	2.86
4	0.438	0.3698	2.97
5	0.503	0.3759	3.02
6	0.569	0.3907	3.13
7	0.637	0.3999	3.20
8	0.681	0.4015	3.24
9	0.734	0.4170	3.34
10	0.797	0.4238	3.40
11	0.845	0.4299	3.44
12	0.898	0.4409	3.53
13	0.953	0.4453	3.57
14	1.015	0.4535	3.63
15	1.063	0.4580	3.67
16	1.115	0.4653	3.73
17	1.165	0.4723	3.79
18	1.217	0.4775	3.83
19	1.265	0.4803	3.85
20	1.332	0.4854	3.89
21	1.394	0.4962	3.98

BAZIN'S SERIES NO. 149

1	0.248	0.3158	2.55
2	0.317	0.3217	2.58
3	0.390	0.3322	2.67
4	0.455	0.3410	2.73
5	0.521	0.3513	2.82
6	0.585	0.3607	2.89
7	0.653	0.3705	2.97
8	0.705	0.3740	3.00
9	0.766	0.3841	3.08
10	0.818	0.3946	3.16
11	0.882	0.4030	3.23
12	0.942	0.4119	3.30
13	0.999	0.4193	3.36
14	1.051	0.4231	3.39
15	1.103	0.4308	3.45
16	1.165	0.4353	3.49
17	1.209	0.4396	3.52
18	1.281	0.4447	3.57
19	1.330	0.4492	3.60
20	1.385	0.4527	3.63
21	1.446	0.4580	3.67

BAZIN'S SERIES NO. 150

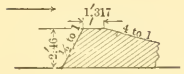


1	0.248	0.3153	2.53
2	0.323	0.3297	2.65
3	0.379	0.3466	2.78
4	0.459	0.3515	2.82
5	0.512	0.3631	2.91
6	0.586	0.3692	2.96
7	0.637	0.3832	3.07
8	0.698	0.3892	3.12
9	0.751	0.3967	3.18
10	0.814	0.4063	3.26
11	0.869	0.4127	3.31
12	0.928	0.4203	3.37
13	0.982	0.4261	3.42
14	1.043	0.4328	3.47
15	1.095	0.4381	3.51
16	1.152	0.4439	3.56
17	1.215	0.4468	3.58
18	1.259	0.4529	3.63
19	1.315	0.4553	3.65
20	1.323	0.4555	3.65
21	1.380	0.4592	3.68
22	1.439	0.4645	3.73

	No. of experiment	h=observed depth on crest, in feet.	m=coefficient of discharge.	$m\sqrt{2g}$
BAZIN'S SERIES NO. 151	1	0.201	0.3373	2.71
	2	0.240	0.3508	2.81
	3	0.307	0.3476	2.79
	4	0.391	0.3473	2.79
	5	0.445	0.3641	2.92
	6	0.514	0.3679	2.95
	7	0.537	0.3720	2.98
	8	0.573	0.3807	3.05
	9	0.613	0.3859	3.09
	10	0.695	0.3990	3.20
	11	0.756	0.4047	3.21
	12	0.800	0.4120	3.30
	13	0.826	0.4131	3.31
	14	0.867	0.4191	3.36
	15	0.921	0.4226	3.39
	16	0.975	0.4311	3.46
	17	1.027	0.4373	3.56
	18	1.090	0.4395	3.52
	19	1.112	0.4459	3.57
	20	1.140	0.4493	3.60
	21	1.209	0.4499	3.61
	22	1.248	0.4546	3.64
	23	1.314	0.4587	3.68
	24	1.352	0.4629	3.71
	25	1.416	0.4669	3.75
BAZIN'S SERIES NO. 153	1	0.237	0.3408	2.73
	2	0.301	0.3458	2.77
	3	0.372	0.3476	2.79
	4	0.373	0.3534	2.83
	5	0.440	0.3613	2.90
	6	0.505	0.3649	2.93
	7	0.576	0.3747	3.00
	8	0.637	0.3827	3.07
	9	0.696	0.3866	3.10
	10	0.701	0.3863	3.10
	11	0.760	0.3930	3.15
	12	0.762	0.3940	3.16
	13	0.814	0.3996	3.20
	14	0.879	0.4057	3.25
	15	0.937	0.4107	3.29
	16	0.993	0.4171	3.34
	17	1.001	0.4159	3.33
	18	1.055	0.4237	3.40
	19	1.102	0.4253	3.41
	20	1.170	0.4318	3.46
	21	1.226	0.4341	3.48
	22	1.290	0.4379	3.51
	23	1.289	0.4392	3.52
	24	1.347	0.4408	3.53
	25	1.404	0.4463	3.58
	26	1.436	0.4466	3.58
BAZIN'S SERIES NO. 154	1	0.236	0.3370	2.70
	2	0.308	0.3421	2.74
	3	0.373	0.3521	2.83
	4	0.447	0.3554	2.85
	5	0.508	0.3677	2.95
	6	0.577	0.3701	2.97
	7	0.643	0.3787	3.04
	8	0.706	0.3826	3.07
	9	0.760	0.3950	3.17
	10	0.823	0.3987	3.20
	11	0.888	0.3995	3.20
	12	0.946	0.4043	3.24
	13	1.011	0.4095	3.28
	14	1.075	0.4123	3.31
	15	1.138	0.4186	3.36
	16	1.195	0.4204	3.37
	17	1.250	0.4245	3.40
	18	1.310	0.4282	3.43
	19	1.370	0.4298	3.45
	20	1.430	0.4334	3.48

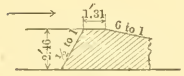
	No. of experiment	h = observed depth on crest, in feet.	m = coefficient of discharge.	$m\sqrt{2g}$
BAZIN'S SERIES NO. 156	1	0.215	0.3441	2.75
	2	0.311	0.3488	2.79
	3	0.382	0.3544	2.83
	4	0.446	0.3611	2.90
	5	0.508	0.3624	2.91
	6	0.576	0.3675	2.95
	7	0.638	0.3758	3.01
	8	0.703	0.3821	3.06
	9	0.764	0.3860	3.09
	10	0.834	0.3897	3.12
	11	0.888	0.3958	3.17
	12	0.956	0.4035	3.24
	13	1.018	0.4021	3.22
	14	1.073	0.4119	3.30
	15	1.138	0.4108	3.28
	16	1.203	0.4126	3.31
	17	1.269	0.4161	3.33
	18	1.340	0.4194	3.36
	19	1.394	0.4195	3.36
	20	1.457	0.4227	3.38
BAZIN'S SERIES NO. 157	1	0.211	0.3282	2.63
	2	0.321	0.3238	2.60
	3	0.395	0 3280	2.63
	4	0.475	0.3299	2.65
	5	0.547	0.3288	2.64
	6	0.624	0.3317	2.66
	7	0.696	0.3371	2.70
	8	0.765	0.3403	2.73
	9	0.826	0.3470	2.78
	10	0.893	0.3498	2.81
	11	0.957	0.3574	2.87
	12	1.030	0.3591	2.88
	13	1.093	0.3655	2.93
	14	1.156	0.3698	2.97
	15	1.216	0.3765	3.02
	16	1.277	0.3801	3.05
	17	1.337	0.3855	3.09
	18	1.394	0.3883	3.11
	19	1.456	0.3959	3.17
	20	1.471	0.3989	3.20
BAZIN'S SERIES NO. 158	1	0.234	0.3479	2.79
	2	0.342	0.3389	2.72
	3	0.383	0.3463	2.77
	4	0.457	0.3479	2.79
	5	0.530	0.3499	2.81
	6	0.609	0.3524	2.82
	7	0.672	0.3559	2.86
	8	0.733	0.3606	2.90
	9	0.799	0.3633	2.91
	10	0.860	0.3682	2.95
	11	0.930	0.3744	3.00
	12	0.984	0.3799	3.04
	13	1.055	0.3866	3.10
	14	1.125	0.3889	3.12
	15	1.179	0.3923	3.14
	16	1.243	0.3983	3.19
	17	1.297	0.4025	3.22
	18	1.361	0.4066	3.25
	19	1.412	0.4112	3.30
	20	1.457	0.4142	3.32

BAZIN'S SERIES NO. 159



No. of experiment	h = observed depth on crest, in feet.	m = coefficient of discharge.	$m\sqrt{2g}$
1	0.234	0.3333	2.68
2	0.304	0.3121	2.75
3	0.379	0.3515	2.82
4	0.387	0.3507	2.82
5	0.457	0.3498	2.81
6	0.516	0.3620	2.91
7	0.526	0.3544	2.84
8	0.599	0.3519	2.82
9	0.664	0.3571	2.87
10	0.670	0.3527	2.83
11	0.735	0.3591	2.88
12	0.797	0.3662	2.94
13	0.801	0.3727	2.99
14	0.876	0.3665	2.94
15	0.935	0.3654	2.93
16	0.994	0.3751	3.01
17	1.068	0.3777	3.03
18	1.125	0.3869	3.10
19	1.145	0.3798	3.05
20	1.198	0.3839	3.08
21	1.261	0.3873	3.11
22	1.320	0.3935	3.15
23	1.332	0.3906	3.13
24	1.389	0.3942	3.14
25	1.445	0.3978	3.19
26	1.456	0.3977	3.19

BAZIN'S SERIES NO. 160



1	0.451	0.3505	2.81
2	0.522	0.3512	2.82
3	0.593	0.3541	2.84
4	0.663	0.3589	2.88
5	0.735	0.3598	2.89
6	0.798	0.3629	2.91
7	0.863	0.3641	2.92
8	0.930	0.3701	2.97
9	0.998	0.3731	2.94
10	1.074	0.3772	3.02
11	1.129	0.3784	3.03
12	1.193	0.3818	3.06
13	1.254	0.3842	3.08
14	1.326	0.3868	3.10
15	1.389	0.3919	3.14
16	1.457	0.3937	3.16

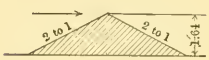
BAZIN'S SERIES NO. 161

1	0 298	0.5373	4.31
2	0.351	0.5357	4.30
3	0.413	0.5308	4.26
4	0.472	0.5272	4.23
5	0.529	0.5259	4.22
6	0.581	0.5297	4.25
7	0.639	0.5289	4.24
8	0.693	0.5314	4.26
9	0.750	0.5331	4.28
10	0.803	0.5374	4.31
11	0.864	0.5373	4.31
12	0.919	0.5385	4.32
13	0.960	0.5405	4.34
14	0.992	0.5359	4.30
15	1.019	0.5374	4.31
16	1.056	0.5340	4.28
17	1.083	0.5323	4.27
18	1.118	0.5282	4.24
19	1.157	0.5196	4.17
20	1.187	0.5188	4.16
21	1.225	0.5139	4.12
22	1.263	0.5102	4 09
23	1.289	0.5118	4.11
24	1.326	0.5087	4.08
25	1.359	0.5086	4.08

BAZIN'S SERIES NO. 163

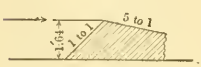
No. of experiment.	h = observed depth on crest, in feet.	m = coefficient of discharge.	$m\sqrt{2g}$
1	0.184	0.4746	3.81
2	0.244	0.4781	3.83
3	0.303	0.4786	3.84
4	0.366	0.4775	3.83
5	0.423	0.4779	3.83
6	0.486	0.4759	3.82
7	0.536	0.4816	3.86
8	0.593	0.4917	3.94
9	0.653	0.4880	3.91
10	0.702	0.4993	4.01
11	0.769	0.4966	3.98
12	0.827	0.5017	4.02
13	0.882	0.5006	4.02
14	0.949	0.5038	4.04
15	0.998	0.5062	4.06
16	1.056	0.5060	4.06
17	1.114	0.5052	4.05
18	1.171	0.5071	4.07
19	1.231	0.5076	4.07
20	1.285	0.5129	4.12
21	1.339	0.5199	4.17

BAZIN'S SERIES NO. 164



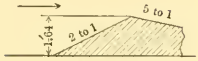
1	0.244	0.4811	3.86
2	0.305	0.4861	3.90
3	0.367	0.4821	3.87
4	0.425	0.4867	3.90
5	0.482	0.4828	3.87
6	0.539	0.4826	3.87
7	0.592	0.4913	3.94
8	0.651	0.4915	3.94
9	0.702	0.4948	3.97
10	0.766	0.4990	4.00
11	0.817	0.5021	4.03
12	0.877	0.5046	4.05
13	0.939	0.5070	4.07
14	0.993	0.5111	4.10
15	1.052	0.5093	4.09
16	1.115	0.5136	4.12
17	1.162	0.5151	4.13
18	1.219	0.5172	4.15
19	1.277	0.5205	4.18
20	1.330	0.5217	4.19

BAZIN'S SERIES NO. 165



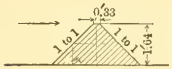
1	0.337	0.4435	3.56
2	0.401	0.4447	3.56
3	0.464	0.4413	3.56
4	0.528	0.4133	3.55
5	0.593	0.4413	3.54
6	0.656	0.4422	3.54
7	0.720	0.4422	3.54
8	0.783	0.4133	3.55
9	0.843	0.4461	3.58
10	0.904	0.4506	3.61
11	0.969	0.4528	3.63
12	1.029	0.4531	3.63
13	1.090	0.4538	3.64
14	1.153	0.4553	3.65
15	1.217	0.4563	3.66
16	1.279	0.4584	3.68
17	1.341	0.4583	3.68
18	1.401	0.4601	3.69
19	1.448	0.4648	3.73

BAZIN'S SERIES NO. 166



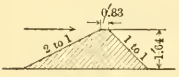
No. of experiment	h = observed depth on crest, in feet.	m = coefficient of discharge.	$m\sqrt{2g}$
1	0.330	0.4563	3.66
2	0.393	0.4548	3.65
3	0.455	0.4536	3.64
4	0.518	0.4538	3.64
5	0.575	0.4579	3.67
6	0.638	0.4579	3.67
7	0.699	0.4583	3.68
8	0.761	0.4596	3.69
9	0.821	0.4625	3.71
10	0.880	0.4663	3.74
11	0.917	0.4677	3.75
12	1.008	0.4681	3.76
13	1.066	0.4699	3.77
14	1.127	0.4714	3.78
15	1.187	0.4723	3.79
16	1.247	0.4731	3.80
17	1.308	0.4746	3.81
18	1.372	0.4748	3.81

BAZIN'S SERIES NO. 167

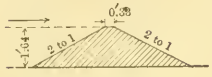
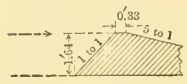
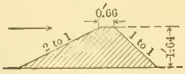


1	0.341	0.4087	3.28
2	0.398	0.4256	3.41
3	0.453	0.4367	3.50
4	0.509	0.4541	3.64
5	0.565	0.4577	3.67
6	0.616	0.4679	3.75
7	0.671	0.4753	3.81
8	0.725	0.4823	3.87
9	0.781	0.4917	3.94
10	0.828	0.4975	3.99
11	0.885	0.5046	4.05
12	0.930	0.5127	4.11
13	0.987	0.5176	4.15
14	1.044	0.5213	4.19
15	1.084	0.5293	4.25
16	1.142	0.5319	4.27
17	1.193	0.5349	4.29
18	1.245	0.5396	4.33
19	1.304	0.5431	4.36
20	1.354	0.5437	4.36

BAZIN'S SERIES NO. 168

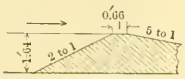


1	0.331	0.4137	3.32
2	0.392	0.4286	3.44
3	0.444	0.4435	3.56
4	0.503	0.4505	3.61
5	0.560	0.4592	3.68
6	0.619	0.4648	3.73
7	0.669	0.4733	3.80
8	0.732	0.4740	3.80
9	0.788	0.4801	3.85
10	0.839	0.4877	3.91
11	0.898	0.4915	3.94
12	0.954	0.4949	3.97
13	1.009	0.4999	4.01
14	1.059	0.5058	4.06
15	1.117	0.5080	4.08
16	1.168	0.5122	4.11
17	1.220	0.5189	4.16
18	1.275	0.5180	4.16
19	1.338	0.5205	4.18
20	1.386	0.5252	4.22

	No. of experiment.	h=observed depth on crest, in feet.	m=coefficient of discharge.	$m\sqrt{2g}$
BAZIN'S SERIES NO. 170 	1	0.351	0.4184	3.35
	2	0.411	0.4300	3.45
	3	0.468	0.4364	3.50
	4	0.529	0.4488	3.60
	5	0.585	0.4488	3.60
	6	0.642	0.4576	3.67
	7	0.701	0.4632	3.72
	8	0.753	0.4689	3.76
	9	0.811	0.4753	3.81
	10	0.871	0.4796	3.85
	11	0.926	0.4821	3.87
	12	0.981	0.4895	3.93
	13	1.009	0.4928	3.95
	14	1.060	0.4948	3.97
	15	1.115	0.4987	4.00
	16	1.174	0.5027	4.03
BAZIN'S SERIES NO. 172 	1	0.356	0.3872	3.11
	2	0.419	0.4025	3.23
	3	0.482	0.4039	3.24
	4	0.539	0.4143	3.32
	5	0.602	0.4175	3.35
	6	0.661	0.4223	3.39
	7	0.722	0.4255	3.41
	8	0.779	0.4333	3.48
	9	0.838	0.4368	3.50
	10	0.906	0.4411	3.54
	11	0.962	0.4436	3.56
	12	1.026	0.4460	3.58
	13	1.086	0.4482	3.59
	14	1.152	0.4499	3.61
	15	1.204	0.4558	3.66
	16	1.267	0.4586	3.69
	17	1.325	0.4607	3.70
	18	1.377	0.4670	3.75
	19	1.449	0.4645	3.73
BAZIN'S SERIES NO. 175 	1	0.360	0.3950	3.17
	2	0.421	0.4061	3.25
	3	0.487	0.4054	3.25
	4	0.548	0.4119	3.30
	5	0.608	0.4220	3.38
	6	0.667	0.4281	3.43
	7	0.726	0.4337	3.48
	8	0.782	0.4396	3.52
	9	0.837	0.4482	3.59
	10	0.894	0.4554	3.65
	11	0.949	0.4607	3.70
	12	1.007	0.4649	3.73
	13	1.061	0.4710	3.78
	14	1.123	0.4747	3.81
	15	1.174	0.4790	3.84
	16	1.230	0.4811	3.86
	17	1.287	0.4845	3.89
	18	1.347	0.4869	3.91
	19	1.387	0.4908	3.94

	No. of experiment	h = observed depth on crest, in feet.	m = coefficient of discharge.	$m\sqrt{2g}$
BAZIN'S SERIES NO. 176	1	0.237	0.3439	2.76
	2	0.296	0.3413	2.74
	3	0.365	0.3636	2.92
	4	0.439	0.3684	2.95
	5	0.494	0.3795	3.04
	6	0.565	0.3867	3.10
	7	0.618	0.3977	3.19
	8	0.682	0.4021	3.22
	9	0.733	0.4104	3.29
	10	0.797	0.4175	3.35
	11	0.861	0.4251	3.41
	12	0.910	0.4296	3.15
	13	0.974	0.4376	3.51
	14	1.027	0.4409	3.53
	15	1.088	0.4449	3.57
	16	1.139	0.4518	3.62
	17	1.196	0.4547	3.65
	18	1.248	0.4586	3.68
	19	1.303	0.4652	3.73
	20	1.355	0.4676	3.75
	21	1.420	0.4727	3.80
BAZIN'S SERIES NO. 177	1	0.367	0.3863	3.10
	2	0.433	0.3872	5.18
	3	0.497	0.3993	3.20
	4	0.556	0.4050	3.25
	5	0.617	0.4127	3.31
	6	0.680	0.4180	3.35
	7	0.739	0.4242	3.40
	8	0.794	0.4309	3.46
	9	0.850	0.4389	3.52
	10	0.906	0.4182	3.59
	11	0.965	0.4562	3.66
	12	1.019	0.4621	3.71
	13	1.077	0.4658	3.74
	14	1.129	0.4716	3.79
	15	1.184	0.4753	3.81
	16	1.243	0.4779	3.83
	17	1.296	0.4840	3.88
	18	1.346	0.4867	3.91
	19	1.386	0.4924	3.95
BAZIN'S SERIES NO. 179	1	0.364	0.3930	3.15
	2	0.431	0.3997	3.20
	3	0.490	0.4054	3.25
	4	0.558	0.4091	3 28
	5	0.614	0.4180	3.35
	6	0.674	0.4241	3.49
	7	0.732	0.4277	3.43
	8	0.790	0.4331	3.47
	9	0.851	0.4399	3.53
	10	0.908	0.4480	3.59
	11	0.968	0.4536	3.64
	12	1.023	0.4588	3.68
	13	1.078	0.4641	3.72
	14	1.134	0.4656	3.71
	15	1.190	0.4703	3.77
	16	1.247	0.4740	3.80
	17	1.298	0.4796	3.85
	18	1.357	0.4840	3.88
	19	1.403	0.4884	3.92

BAZIN'S
SERIES NO. 181



No. of experiment.	h=observed depth on crest, in feet.	m=coefficient of discharge.	$m\sqrt{2g}$
1	0.219	0.3789	3.04
2	0.290	0.3858	3.09
3	0.356	0.3863	3.10
4	0.423	0.3900	3.13
5	0.486	0.3993	3.20
6	0.554	0.3970	3.18
7	0.621	0.4030	3.23
8	0.677	0.4092	3.28
9	0.743	0.4105	3.29
10	0.797	0.4153	3.33
11	0.861	0.4210	3.38
12	0.920	0.4267	3.42
13	0.983	0.4311	3.46
14	1.044	0.4345	3.49
15	1.111	0.4394	3.52
16	1.159	0.4413	3.54
17	1.228	0.4447	3.57
18	1.278	0.4484	3.59
19	1.342	0.4518	3.62
20	1.403	0.4534	3.63
21	1.463	0.4578	3.67

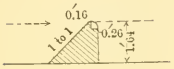
BAZIN'S
SERIES NO. 182

1	0.280	0.3927	3.15
2	0.403	0.4012	3.22
3	0.526	0.4150	3.33
4	0.641	0.4309	3.46
5	0.761	0.4448	3.56
6	0.867	0.4584	3.68
7	0.989	0.4654	3.73
8	1.091	0.4749	3.81
9	1.210	0.4821	3.87
10	1.320	0.4870	3.91
11	1.413	0.4966	3.99

BAZIN'S
SERIES NO. 188

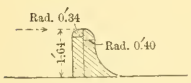
1	0.289	0.3758	3.01
2	0.425	0.3835	3.08
3	0.563	0.3836	3.08
4	0.697	0.3925	3.15
5	0.825	0.3982	3.19
6	0.953	0.4045	3.24
7	1.080	0.4120	3.30
8	1.201	0.4193	3.36
9	1.322	0.4274	3.43
10	1.431	0.4388	3.52
11	1.546	0.4432	3.55

BAZIN'S
SERIES NO. 189



No. of experiment.	h = observed depth on crest, in feet.	m = coefficient of discharge	$m\sqrt{2g}$	
1	0.248	0.4605	3.69	Adhering Nappe.
2	0.300	0.4834	3.88	
3	0.355	0.4967	3.99	
4	0.406	0.5090	4.08	
5	0.457	0.5198	4.17	
6	0.511	0.5331	4.28	
7	0.557	0.5389	4.32	
8	0.607	0.5517	4.43	
9	0 655	0.5563	4.46	
10	0.706	0.5635	4.52	
11	0.706	0.5624	4.51	Nappe wetted underneath.
12	0.756	0.5664	4.54	
13	0.811	0.5706	4.58	
14	0.864	0.5698	4.57	
15	0.923	0.5677	4.55	
16	0.979	0.5681	4.56	
17	1.042	0.5644	4.53	
18	1.097	0.5612	4.50	
19	1.151	0.5629	4.52	
20	1.227	0.5550	4.45	
21	1.274	0.5572	4.47	

BAZIN'S.
SERIES NO. 193



1	0.260	0.4354	3.49	
2	0.317	0.4496	3.61	
3	0.374	0.4635	3.72	
4	0.429	0.4716	3.79	
5	0.478	0.4891	3.92	
6	0.534	0.4924	3.95	
7	0.585	0.5049	4.05	
8	0.634	0.5108	4.10	
9	0.684	0.5238	4.20	
10	0.736	0.5293	4.25	
11	0.780	0.5418	4.35	
12	0.836	0.5480	4.40	
13	0.886	0.5559	4.46	
14	0.932	0.5628	4.52	
15	0.979	0.5644	4.53	
16	1.035	0.5710	4.58	
17	1.093	0.5675	4.55	
18	1.148	0.5663	4.54	
19	1.201	0.5695	4.57	
20	1.266	0.5665	4.54	

General Résumé of Bazin's Experiments.

This chapter has also been translated from Bazin's final paper in *Annales des Ponts et Chaussées* for 1898.*

Classification of the Different Species of Nappes.—When water is discharged through a submerged orifice, it is well known that the discharge Q varies according to the nature of the orifice. The coefficient m, in the formula $Q = m\, c \sqrt{2\, g\, h}$, must be determined for each particular case. This coefficient varies, however, only within relatively narrow limits.

The conditions in the case of a weir are much more complex. The influence of the shape (width of crest, degree of inclination of the up-stream and down-stream faces of the weir, etc.) always enters, but other factors conspire to make the value of the coefficient m, in the formula $Q = m\, l\, h \sqrt{2\, g\, h}$, vary within much wider limits than for discharge through an orifice.

The sloping nappe of a weir differs from the vein issuing from an orifice in that it may assume a variety of perfectly distinct forms. These forms constitute, in reality, as many distinct cases, each of which it is necessary to study separately, since, by confusing them one must necessarily expose himself to serious errors. It is proper, at the start, to classify these different forms and to make known their distinctive characteristics.

Free Nappes.—The most simple and definite case is that of a sharp-crested weir without lateral contractions, in which the nappe falls

*Bazin's formulas have been changed, so as to make English measures applicable, by the introduction of a conversion factor when necessary. The mathematical symbols are as follows:

h = head on crest of weir, in feet;
u = mean velocity in channel above the weir, in feet;
a = a constant (replaces α used by Bazin);
K = a constant;
g = acceleration of gravity, = 32.2;
l = length of crest of weir, in feet;
n = a constant (replaces μ used by Bazin);
p = height of crest of weir above bottom of channel, in feet;
m = coefficient of discharge over a given weir;
m' = corresponding coefficient for a standard weir;
Q = discharge, in cubic feet per second;
h_f = difference of elevation of crest of weir and of water on the down-stream side, in feet, when the latter is below the crest of the weir;
h_e = difference of elevation of crest of weir and of water on the down-stream side, in feet, when the weir is drowned;
c = width of crest of a flat-crested weir, in feet;
R = radius of curvature of a weir rounded at the back, in feet;
$z = h - h_e$, in feet.

freely in the air, its lower surface always subject to the pressure of the atmosphere. The lateral contraction may be suppressed by making the length of the weir equal to the width of the canal leading to it, which is supposed to have vertical walls. Immediately below the crest, these walls should be constructed in such a manner as to permit free access of air beneath the nappe, which then represents a portion of a nappe of indefinite length. The resulting phenomena of flow are perfectly constant; the nappe, independent of any influence of the water below the weir, permits of a precise determination of the coefficient m.

Depressed Nappes and Nappes Wetted Underneath.—When the walls of the canal in which the weir is placed are not constructed in such a manner as to maintain free access of air beneath the nappe, the phenomena of discharge become more complicated, and the form of the nappe, remarkably constant in the case where it falls freely over the weir, undergoes considerable modification, according to the amount of discharge.

When the head does not exceed a certain limit, the nappe remains detached from the face of the weir, imprisoning underneath it a volume of air at a pressure inferior to that of the external atmosphere. At the same time the water in the space between the foot of the nappe and that of the weir rises to a level above that of the stream below the weir. We have, then, a species of free nappe, modified and depressed by the excess of external pressure. The discharge over such a sharp-crested weir is, for equal heads, slightly greater than over a true free nappe. The difference increases as the volume of the imprisoned air diminishes.

Depressed nappes are not very stable; the accidental entrance of air from time to time produces variations in both the interior pressure and the discharge. When the air has entirely disappeared, the nappe takes a more definite form, which may be designated as wetted underneath. The overflowing nappe encloses a small portion of turbulent water, which does not partake of the translatory movement of the vein, properly speaking.

The occurrence of a nappe wetted underneath may be independent of the level of the water on the down-stream side of the weir; or, on the contrary, it may be influenced by this level, every modification of which then reacts on the discharge over the weir.

The former is the case when the overfall is followed by a rapid which terminates in an abrupt ressault, below which the flow takes place in accordance with the condition of the channel. The position of the ressault is without influence on the discharge, provided it does not enclose the foot of the nappe.

In the second case, that is to say, when the foot of the nappe is more or less enclosed in the water in the channel below the weir, it is not easy to separate the influence of the water in the channel, for the discharge may be modified, although this water does not rise to the level of the crest of the weir.

Undulating Nappes.—When the level of the water in the channel below the weir is raised to the height of the weir crest, the wetted nappe retains its form as long as the difference in level, or fall of the water from the up-stream to the down-stream side of the weir, does not descend below a certain limit. Its characteristic profile persists, although in part concealed by its immersion in the channel on the down-stream side, but, where the fall or difference in level is progressively diminished, a moment comes when the nappe returns suddenly to the surface, undulating in the meanwhile. This change, although very apparent, does not exert any important influence on the value of the coefficient of discharge.

Adhering Nappes.—The forms of nappes thus far considered are those most ordinarily encountered. Another form exists, which appears under certain conditions, in which the nappe, instead of enfolding a small mass of turbulent water having no translatory movement, as in the case of the nappe wetted underneath, is, on the contrary, in close contact with the face of the weir. It presents then, in certain cases, some interesting particulars, and to this remarkable form there often corresponds a considerable increase of the coefficient of discharge.

The *ensemble* of the phenomena of discharge is very complex, and one cannot often determine the discharge of a weir with precision without previously knowing under which particular form of nappe the discharge took place. Taking, for example, a sharp-crested weir 0.75 m. (2.46 ft.) high, we have shown that for the same head of 0.20 m. (0.656 ft.) the nappe may assume four very distinct forms, to which correspond the following different values of the coefficient m in the formula $Q = m\,l\,h\,\sqrt{2\,g\,h}$:

	m.	$m\sqrt{2g}$.
(1) Free nappe, under surface always subjected to atmospheric pressure	0.433	3.47
(2) Depressed nappe, imprisoning a certain volume of air at a pressure below that of the atmosphere	0.460	3.69
(3) Nappe wetted underneath, no air imprisoned, level of water on down-stream side 0.125 m. (0.42 ft.) below crest of weir	0.497	3.99
(4) Adhering nappe, the ressault being at a distance from the foot of the nappe, which is completely exposed	0.554	4.45

Sharp-Crested Weirs.

Free Nappes.—When the nappe, in flowing over a sharp-crested weir, has its lower surface in free communication with the atmosphere, the only element which modifies the coefficient m is the mean velocity u in the channel leading to the weir. In order to take this into consideration in the formula, h is replaced by $h + a\frac{u^2}{2g}$. The formula becomes then, representing by n the modified coefficient m,

$$Q = n\,l\left(h + a\frac{u^2}{2g}\right)\sqrt{2g\left(h + a\frac{u^2}{2g}\right)}$$

$$= n\,l\,h\sqrt{2g\,h}\left(1 + a\frac{u^2}{2g\,h}\right)^{\frac{3}{2}}$$

or, approximately, considering that $\frac{u^2}{2g\,h}$ is a fraction rarely exceeding a few centimeters,

$$Q = n\,l\,h\sqrt{2g\,h}\left(1 + \frac{3}{2}\,a\frac{u^2}{2g\,h}\right) \quad \ldots\ldots\ldots\ldots \quad (1)$$

This expression is not in a form convenient for use, since the velocity u depends on the discharge to be determined. If p be used to designate the height of the weir above the bottom of the channel, the wetted section of the canal is $l\,(h + p)$, and we have

$$\frac{u^2}{2g} = \frac{Q^2}{2g\,l^2(h+p)^2}$$

or, simply replacing Q by its value $m\,l\,h\sqrt{2g\,h}$, we have,

$$\frac{u^2}{2g\,h} = m^2\left(\frac{h}{h+p}\right)^2$$

Letting, for short $\left(\frac{3}{2}\, a\, m^2\right) = K$, the above expression takes the more practical form

$$Q = n\left[1 + K\left(\frac{h}{h+p}\right)^2\right] l\, h\, \sqrt{2gh} \ldots\ldots\ldots \quad (2)$$

We have determined the coefficients a, K and n by comparative experiments on five weirs of different heights. a and K are not perfectly constant, but one may take, as a mean, $a = \frac{5}{3}$; $K = 0.55$. As to n, its value, which corresponds to the limiting case of no velocity of approach, cannot be measured directly, since it is impossible to completely suppress this velocity. Its influence, however, becomes insignificant in a high weir. The coefficient n decreases slowly as the head increases, as shown below:

Head, in feet = 0.164, 0.328, 0.656, 0.984, 1.312, 1.640.

Corresponding values of n = 0.448, 0.432, 0.421, 0.417, 0.414, 0.412.

When h is over 0.328 ft. its value is represented with sufficient precision by the formula,

$$n = 0.405 + \left(\frac{0.00984}{h}\right) \ldots\ldots\ldots\ldots\ldots \quad (3)$$

Adopting for K the value 0.55, Formula (2) becomes

$$m = n\left[1 + 0.55\left(\frac{h}{h+p}\right)^2\right] \ldots\ldots\ldots\ldots \quad (4)$$

In most cases, where the head falls between 0.10 m. (0.328 ft.) and 0.30 m (0.984 ft.), n may be taken at the constant value 0.425, and taking $K = \frac{1}{2}$ simply, the expression for m becomes

$$\left.\begin{aligned} m &= 0.425\left[1 + \frac{1}{2}\left(\frac{h}{h+p}\right)^2\right] \\ &= 0.425 + 0.212\left(\frac{h}{h+p}\right)^2 \end{aligned}\right\} \ldots\ldots\ldots \quad (5)$$

which will answer for all practical cases where errors of 2 to 3% are permissible. It is this coefficient of discharge m, perfectly determined by the head h and the height p of the weir, which has been used for comparison. Instead of considering on the other weirs the absolute values of the coefficient m, we have compared them with the coefficient m' for a free nappe, for the same head on a sharp-crested weir of the same height.

This substitution of the ratio $\frac{m}{m}$, for the absolute values of m eliminates, in a large measure, at least, the influence of velocity of approach, and facilitates greatly the discussion of results.

In what precedes, the up-stream face of the weir has been supposed to be a vertical plane. If it is inclined, the values of the coefficient m will be modified. The coefficient is diminished when the plane of the dam is inclined up stream, but, if, on the contrary, the plane of the dam is inclined down stream, the coefficient increases to a maximum which corresponds nearly to an inclination of 30° (equals a slope of $1\frac{3}{4}:1$ on the back). The ratio between the coefficients corresponding to two different inclinations is sensibly constant for all heads, so that one may obtain the coefficient m for a sharp-crested weir at any inclination by multiplying by a constant, or modulus, the corresponding coefficient for a vertical weir, as is indicated in Table No. 3.

This ratio increases regularly from an inclination of 45° toward the up-stream side, to approximately 30° toward the down-stream side where the maximum occurs, from which point the discharge does not take place in the normal manner, since the liquid vein in its passage over the crest, instead of being freely contracted, is guided by the incline of the weir on which it rests in immediate contact.

TABLE No. 3.

		Modulus by which to multiply the coefficient for a vertical weir.
Up-stream inclination of the weir.	1 horizontal to 1 vertical	0.93
	2 " " 3 "	0.94
	1 " " 3 "	0.96
Vertical weir		1.00
Down-stream inclination of the weir.	1 horizontal to 3 vertical	1.04
	2 " " 3 "	1.07
	1 " " 1 "	1.10
	2 " " 1 "	1.12
	4 " " 1 "	1.09

Sharp-Crested Weirs. Nappes Depressed and Wetted Underneath.— When the air is not admitted freely underneath the nappe, the phenomena become more complicated. The nappe may be either depressed, as a result of air being imprisoned underneath at less than atmospheric pressure, or it may be wetted underneath without

containing any air. The discharge for a depressed nappe is slightly in excess of that for a free nappe. The difference may rise to nearly 10% at the moment when the nappe, the confined air being nearly exhausted, is at the point of assuming the form of a nappe wetted underneath. The accidental entrance of air from time to time may vary the discharge a little. The wetted nappes are more uniform. It is important to distinguish two cases according as the ressault, which is produced below the nappe, is at a distance from its foot, or partly encloses it.

First Case. Ressault at a Distance.—The coefficient m may be deduced from the coefficient m' for a free nappe by the relation

$$m = m' \left(0.878 + 0.123 \frac{p}{h}\right) \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

The ratio $\frac{p}{h}$ can only have certain values, as experience has shown that it does not exceed 2.5, because the form of nappe wetted underneath does not continue if the head is less than 0.4 p. For the maximum value $\frac{p}{h} = 2.5$, we have, very nearly, $m = 1.20\ m'$; and when $h = p$, m becomes sensibly equal to m'. Finally, m is a little greater than m' when h surpasses p.

If the above formula be applied to weirs of different heights, it may be shown that for the same value of $\frac{p}{h}$ the absolute values of m do not differ greatly from those given by the equation

$$m = 0.470 + 0.0075 \frac{p^2}{h^2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

which permits us to find an absolute value of the coefficient m without using the ratio $\frac{m}{m'}$.

Second Case. The Ressault Enclosing Part of the Nappe.—It is necessary to take into account the level of the water below the weir, and, if we designate by h_l the difference of level of the crest of the weir and of the water below, the value of m becomes

$$m = m' \left[1.06 + 0.16 \left(\frac{h_l}{p} - 0.05\right) \frac{p}{h}\right] \ldots\ldots\ldots\ldots (8)$$

In this formula, h_l is to be taken as minus when the level of the water on the down-stream side is below the crest, and as plus when it is above the crest. The formula can only be applied within certain limits of

h_l. If the difference in the level of the water above and below the weir be increased, a moment comes when the ressault is driven back from the foot of the nappe until it ceases to enclose it and changes then to the preceding case. This pushing back of the ressault takes place when the total fall $(h + h_l)$ is approximately equal to $\frac{3}{4}\,p$. That is to say, for a given head h the greatest admissible value of h_l is $\left(\frac{3}{4}\,p - h\right)$. On the other hand, when the head h is insufficient to throw back the ressault, it is necessary that the level below the weir be sufficient to sustain the foot of the nappe and to prevent the introduction of air, which would cause the nappe to return to the depressed form. The preceding formula may be simplified by suppressing the small term, 0.55, in the parentheses, and, for compensation, slightly diminishing the two other coefficients. It then becomes

$$m = m' \left(1.05 + 0.15\,\frac{h_l}{h}\right) \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

Sharp-Crested Weirs. Adhering Nappes.—The nappe may also take, though very rarely, a particular form, the production of which depends on the width of the dam and the form of the upper part supporting the sharp crest. The nappe becomes completely attached to the downstream face of the dam without the interposition of air. The coefficient of discharge then becomes very large and may rise as high as 1.30 m', which corresponds to an absolute value of the coefficient $m = 0.55$ or 0.56. Adhering nappes present curious particulars, but as they only occur exceptionally in practice, we may simply refer to the special studies made of them, included in *Annales des Ponts et Chaussées* for 1891.

Beam Weirs. Free Nappes.—Beam weirs are constructed of square timbers of the same cross-sectional dimensions, placed one upon another to the desired height. The back and front faces of the weir are vertical planes, but the crest, instead of being reduced to a sharp edge, presents a horizontal surface, the width of which equals the thickness of the timbers. This circumstance completely modifies the conditions of discharge, and, while this form of weir is readily constructed, it may, unfortunately, give considerable error in the gaugings.

The free nappes appear under two distinct forms, according as the

nappe is in contact with the horizontal crest, or becomes detached at the back edge in such a manner as to flow over the crest without touching the down-stream edge. In the second case the influence of the flat crest evidently disappears and the discharge is like that over a sharp-crested weir. The nappe assumes this form when the head exceeds twice the width of c of the crest, measured in the direction of discharge, but it may occur whenever the head exceeds $\frac{3}{2}c$. Between these limits the nappe is in a state of instability; it tends to detach itself from the crest and may do so under the influence of any external disturbance, as, for example, the entrance of air or the passage of a floating body over the weir.

When the nappe adheres to the crest, the coefficient m depends chiefly on the ratio $\frac{h}{c}$ and may be represented by the formula,

$$m = m' (0.70 + 0.185 \frac{h}{c}) \quad \ldots\ldots\ldots\ldots\ldots\ldots (10)$$

m varies, as a consequence, very rapidly. We have,

When $\frac{h}{c} = 0.50$	$\frac{m}{m'} = 0.79$	
" $= 1.00$	" $= 0.88$	
" $= 1.50$	" $= 0.98$	} or 1.0 if the nappe is
" $= 2.00$	" $= 1.07$	} detached.

When $\frac{h}{c}$ exceeds 2.00, $\frac{m}{m'} = 1.00$. It will be seen that between $h = \frac{3}{2}c$ and $h = 2c$, $\frac{m}{m'}$ may vary from 0.98 to 1.07, or nearly a tenth in value, or, it may remain constantly equal to unity, according as the nappe is attached to or free from the crest.

Very Wide Crests.—When the width of the crest is considerable, 1 or 2 m., for example, the foregoing formula is still applicable, giving results within a few per cent. The value of $\frac{h}{c}$ then reduces to a few tenths and the ratio $\frac{m}{m'}$ also becomes much smaller, so that m may not exceed 0.35. For example, at a head of 0.45 m. (1.476 ft.) on a weir with a flat crest 2 m. wide, $\frac{m}{m'} = 0.755$, which corresponds to an absolute value of $m = 0.337$. The formula gives $\frac{m}{m'} = 0.732$ and as a result, $m = 0.326$.

Effect of Rounding the Crest at the Back.—A slight rounding of the back edge of the crest very sensibly modifies the discharge. Fteley and Stearns have shown that rounding the back edge of the crest to a radius R augments the discharge by an amount equal to that given by a head increased by 0.7 R. This is equivalent to increasing the coefficient m in the ratio of $h^{\frac{3}{2}}$ to $(h + 0.7\ R)^{\frac{3}{2}}$ or nearly in the ratio of 1 to $1 + \frac{R}{h}$. The radius R in their experiments did not exceed 0.039 ft., and it is clear that this approximate mode of correction will not apply to cases where the radius is notably greater. We have experimented on two weirs, respectively 0.80 m. (2.624 ft.) and 2.00 m. (6.56 ft.) in width, with crests rounded at the back to a radius of 0.10 m. (0.328 ft.), and this modification has had the effect of increasing the discharge 14% on the first of these weirs and 12% on the second. A simple rounding of 1 or 2 cm. radius, such as results from wear on timbers of ordinary dimensions, is by no means negligible from the point of view of the resulting discharge.

A weir with a crest 2 m. wide, and rounded at the back, gave, for the greatest head used in the experiments, $m = 0.373$, a value differing little from that indicated by theory for the case of a nappe flowing in filaments parallel to the horizontal surface of the crest. This hypothesis may not, however, be realized experimentally in more than a very imperfect manner, as the surface of the nappe undulates continually.

Beam Weirs. Nappes Depressed and Wetted Underneath.—The depressed nappes do not differ greatly from the free. The coefficient is at first less than for a free nappe, but approaches it progressively in value and finally exceeds it slightly. It differs in this respect from a sharp-crested weir, for which the coefficient for a depressed nappe is always superior to that for a free nappe. It makes no difference, as to this, whether the nappe clings to the flat crest or is detached from it. In either case the coefficient differs little from that for a sharp-crested weir. The effect of adherence to the flat crest appears again for a nappe wetted underneath, with this added difficulty, that the moment of detachment underneath the water is not apparent, and does not correspond to any constant value of $\frac{h}{c}$. In other words, it may take place either preceding or following the formation of the

nappe wetted underneath. It is necessary, in this regard, to distinguish two cases according as the height p of the dam is greater or less than about five times the width c of the crest. When p is greater than $5\,c$, the nappe detaches itself from the flat crest before it becomes wetted underneath, and in the intermediate state does not differ greatly from that for a sharp-crested weir. When, on the other hand, p is less than $5\,c$, the nappe does not detach itself from the flat crest before becoming wetted underneath, but is very unstable at the moment of this transformation.

So long as the nappe adheres to the crest, this influence predominates, and Formula (10) is most nearly applicable to the nappe wetted underneath. On the other hand, when the nappe is detached from the flat crest, the conditions of discharge approach more nearly those for a sharp crest, to which Formula (6) may be applied. The two formulas give the same value when the head exceeds a certain limit:

$$h_l = \frac{c}{2}\left(1 + \sqrt{\frac{3\,p}{c}}\right) \qquad (a)$$

For heads less than h_l, Formula (10) gives values of m slightly too small, never differing from those of the experiments, however, by more than 3 or 4 per cent. When the head exceeds h_l, one must take recourse to the other formula, although it, likewise, gives values which are too small. The difference, rather more important in this case, attains 8% as a maximum, after which it diminishes rapidly for increased heads. This maximum corresponds to the moment when the nappe is at the point of detaching itself from the flat crest. After it has become detached, the influence of width of crest disappears and Formula (6) applies with a very close degree of approximation.

It will be seen that the flat crest has the effect of doubling each species of nappe, in that two formulas must be applied according as the nappe clings to or is detached from the flat surface.

Weirs with Wide Crests, and Slope on the Faces.—The phenomena of discharge become much more complex for weirs, such as are often found in practice, with slopes of greater or less inclination on the front and back faces. The influence of the flat crest, which exerts itself in a weir built up of square timbers, is joined to that of the slope of the faces. The inclination of the up-stream face, by reduc-

ing the contraction, has the effect of increasing the discharge. While that of the down-stream face has, ordinarily, the same effect as increasing the width of the flat crest, that is to say, it diminishes the discharge. The coefficient m, then, in each particular case, depends not only on the head, but on the width of the crest and the degree of inclination of the faces. It is, therefore, exceedingly variable, and each type demands a special study.

Rounding the back edge of the crest reduces the contraction considerably and may increase the value of m 10 or 15 per cent. Considering, finally, the class of weirs with completely curved profiles, such as are occasionally encountered in hydraulic practice, the value of m may attain a relatively high figure. The coefficients for such cases have not been arranged in comparative tables, but enough particular cases are given to serve as a guide in practice. It is clearly impossible to establish a general formula which will take account of all the elements that enter to affect the discharge.

Drowned Weirs.—We have given, in discussing the experiments on sharp-crested weirs drowned by the water on the down-stream side, two formulas; one of which applies to cases where the weir is not deeply drowned. The other, which is more general in its application, is

$$m = m' \left(1.08 + 0.18 \frac{h_e}{p}\right) \sqrt[3]{\frac{z}{h}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (11)$$

The two formulas mentioned have been so established as to represent in the best possible manner the experiments from which they have been deduced. In cases where a less precise approximation will suffice, Formula (11) may be made applicable, by altering slightly its coefficients, as follows:

$$m = 1.05\, m' \left[1 + \frac{1}{5}\left(\frac{h_e}{p}\right)\right] \sqrt[3]{\frac{z}{h}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (12)$$

This new expression is practically equivalent to the two others and gives the same values within 1 or 2%, except when the ratios $\frac{h}{p}$ and $\frac{h_1}{p}$ are very small. The difference may then be as much as 4 or 5%, but in this case the determination of the coefficient m is always very uncertain.

The effect of drowning is not the same for a wide-crested weir.

Raising the plane of the water below the weir, which in the case of a sharp-crested weir affected the flow on the up-stream side before the water below had reached the height of the crest, does not commence to take effect on a wide-crested weir until after the level of the water on the down-stream side is considerably above the crest; and the greater the width of the crest, the less is its ultimate effect. In our experiments on a crest 2 m. in width we have shown that the water on the down-stream side must rise to a height above the crest equal to $\frac{5}{6}$ of the head h before it affects the level on the up-stream side.

When the plane surface, which forms the crest of a weir, is very wide, it constitutes a sort of channel, and, in a measure, as the length of this channel is increased, the conditions of discharge depart from those which pertain to a weir, properly speaking, and approach those for a channel with a horizontal bottom.

The Cornell University Experiments.

At the beginning of the study of Bazin's work it was the writer's opinion that his coefficients could be fairly extended to depths on the crest of about 4 ft. without material error, and on this basis a number of discharge curves were worked out in the manner to be described. On further study, however, it seemed probable that some of Bazin's Series, especially Nos. 130 and 135 and a few others, might be somewhat too high for deep flows, for the reason that at depths on the crest from 0 up to about 0.6 to 1.0 ft. the nappes were depressed and adherent, and above 0.6 to 1.0 ft. were wetted underneath, thus indicating that probably the conditions of the experiments were such as not to insure the free admission of air beneath the nappes, this condition leading to higher flows than with air freely admitted. Or, on the other hand, as Bazin himself points out, the limit of perfect detachment may not have been reached in his experiments.

Again, Bazin's weirs were constructed with closed fronts, thus offering an opportunity for adhering nappes, while in actual practice, for sections corresponding to Series Nos. 130 and 135, the water generally flows over a lip, the nappe dropping into a free air space below. This general condition is illustrated by the dams shown on several of the illustrative figures following. The conditions at the ends of such dams are such as to usually permit the free admission of air.

CORNELL UNIVERSITY

EXPERIMENT NO. 1

BAZIN'S SERIES NO. 130

MAY 20TH, 1899.

Length of crest 6.58 ft.

Number of experiments, 7.

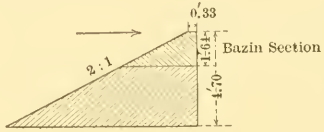
Limiting heads, 1.19 ft. and 4.97 ft.

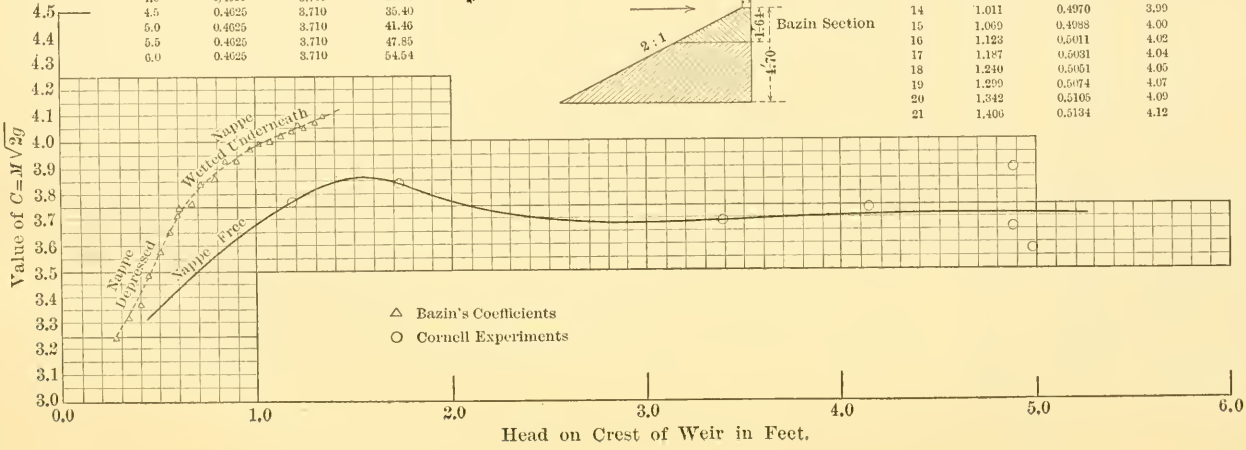
Head, in feet. H	M	$C = M\sqrt{2g}$	Q per foot.
0.5	0.4180	3.355	1.20
1.0	0.4588	3.680	3.44
1.5	0.4764	3.820	6.59
2.0	0.4698	3.770	10.66
2.5	0.4598	3.690	14.59
3.0	0.4585	3.680	19.13
3.5	0.4598	3.690	24.25
4.0	0.4616	3.705	29.68
4.5	0.4625	3.710	35.40
5.0	0.4625	3.710	41.46
5.5	0.4625	3.710	47.85
6.0	0.4625	3.710	54.54

BAZIN'S

SERIES NO. 130

No. of experiment.	h = observed depth on crest, in feet.	m = coefficient of discharge.	$m\sqrt{2g}$	
1	0.274	0.4037	3.24	Nappe depressed or adhering, but unstable.
2	0.336	0.4138	3.32	
3	0.396	0.4213	3.37	
4	0.452	0.4339	3.48	
5	0.510	0.4458	3.57	
6	0.565	0.4546	3.65	
7	0.618	0.4663	3.74	Nappe wetted underneath and attached to flat crest.
8	0.672	0.4694	3.76	
9	0.728	0.4769	3.83	
10	0.785	0.4801	3.85	
11	0.837	0.4882	3.92	
12	0.898	0.4893	3.92	
13	0.956	0.4949	3.97	
14	1.011	0.4970	3.99	
15	1.069	0.4988	4.00	
16	1.123	0.5011	4.02	
17	1.187	0.5031	4.04	
18	1.240	0.5051	4.05	
19	1.299	0.5074	4.07	
20	1.342	0.5105	4.09	
21	1.406	0.5134	4.12	





CORNELL UNIVERSITY

EXPERIMENT NO. 2.

BAZIN'S SERIES NO. 135.

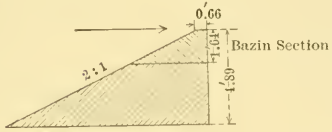
MAY 24TH, 1899.

Length of crest 6.58 ft.

Number of experiments, 7.

Limiting heads, 0.341 ft. and 5.05 ft.

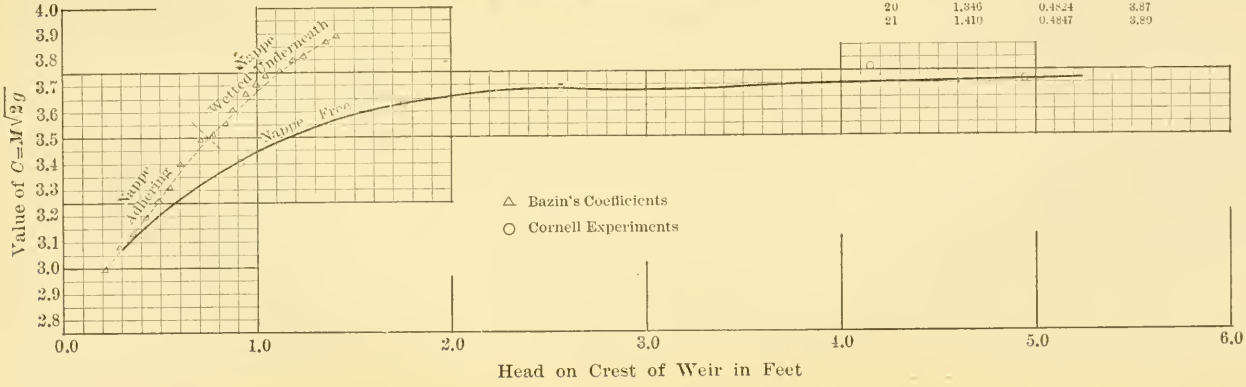
Head, in feet. H	M	$C=M\sqrt{2g}$	Q per foot.
0.5	0.4012	3.220	1.15
1.0	0.4285	3.440	3.44
1.5	0.4474	3.590	6.59
2.0	0.4564	3.663	10.35
2.5	0.4598	3.690	14.59
3.0	0.4585	3.680	19.12
3.5	0.4598	3.690	24.15
4.0	0.4615	3.705	29.65
4.5	0.4625	3.710	35.40
5.0	0.4625	3.710	41.46
5.5	0.4625	3.710	47.84
6.0	0.4625	3.710	54.54



BAZIN'S

SERIES NO. 135.

No. of experiment.	h = observed depth on crest, in feet.	m = coefficient of discharge.	$m\sqrt{2g}$	
1	0.224	0.3727	2.99	Nappe depressed.
2	0.202	0.3830	3.08	
3	0.353	0.3912	3.13	
4	0.422	0.3981	3.19	
5	0.480	0.4071	3.26	
6	0.541	0.4127	3.31	
7	0.597	0.4243	3.40	
8	0.657	0.4264	3.42	Nappe wetted
9	0.712	0.4347	3.49	underneath.
10	0.773	0.4394	3.52	
11	0.829	0.4430	3.55	
12	0.892	0.4496	3.61	
13	0.939	0.4575	3.67	
14	1.011	0.4614	3.70	
15	1.063	0.4657	3.74	
16	1.119	0.4695	3.76	
17	1.175	0.4740	3.80	
18	1.230	0.4757	3.82	
19	1.275	0.4839	3.88	
20	1.346	0.4824	3.87	
21	1.410	0.4847	3.89	



CORNELL UNIVERSITY

EXPERIMENT NO. 3.

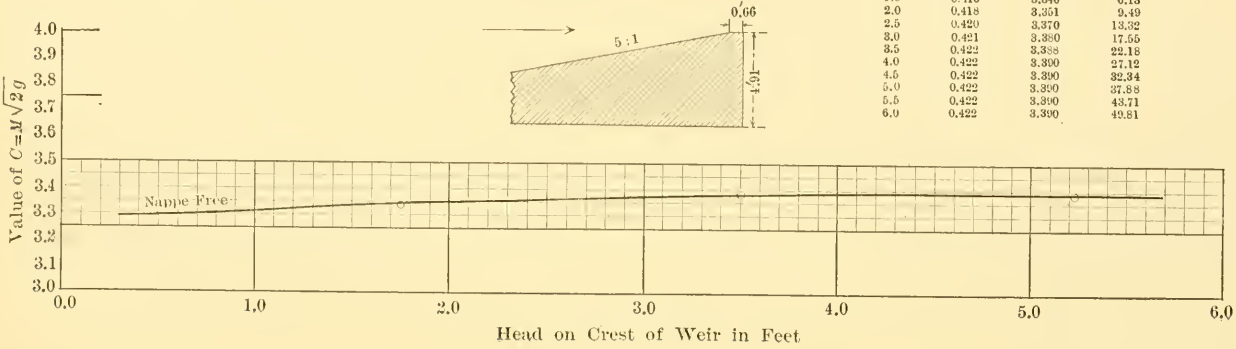
MAY 28TH, 1899.

Length of crest 6.58 ft.

Number of experiments, 3.

Limiting heads, 1.75 ft. and 5.23 ft.

Head, in feet. H	M	$C=M\sqrt{2g}$	Q per foot.
0.5	0.412	3.310	1.18
1.0	0.415	3.330	3.33
1.5	0.416	3.340	6.13
2.0	0.418	3.351	9.49
2.5	0.420	3.370	13.32
3.0	0.421	3.380	17.55
3.5	0.422	3.388	22.18
4.0	0.422	3.390	27.12
4.5	0.422	3.390	32.34
5.0	0.422	3.390	37.88
5.5	0.422	3.390	43.71
6.0	0.422	3.390	49.81



CORNELL UNIVERSITY

EXPERIMENT No. 4.

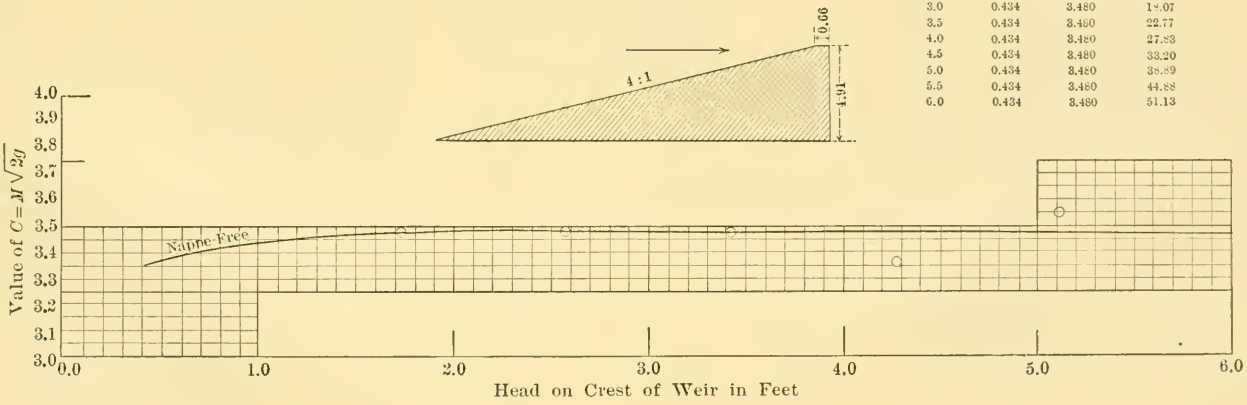
MAY 26TH, 1899.

Length of crest 6.58 ft.

Number of experiments, 6.

Limiting heads, 0.917 ft. and 5.11 ft.

Head, in feet. H	M	$C=M\sqrt{2g}$	Q per ft.
0.5			
1.0	0.429	3.440	3.44
1.5	0.432	3.465	6.36
2.0	0.434	3.480	9.83
2.5	0.434	3.485	13.77
3.0	0.434	3.480	18.07
3.5	0.434	3.480	22.77
4.0	0.434	3.480	27.83
4.5	0.434	3.480	33.20
5.0	0.434	3.480	38.89
5.5	0.434	3.480	44.88
6.0	0.434	3.480	51.13



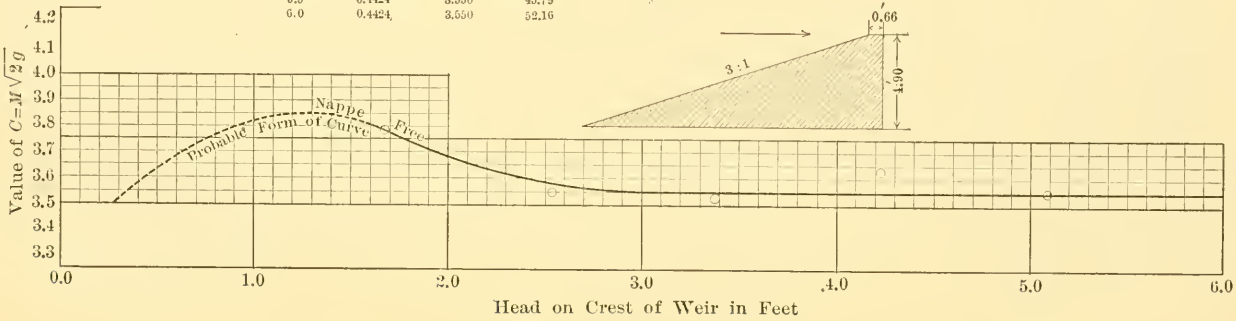
CORNELL UNIVERSITY

EXPERIMENT NO. 5.

MAY 27TH, 1899.

Length of crest 6.58 ft.
Number of experiments. 5.
Limiting heads, 1.689 ft. and 5.085 ft.

Head, in feet. H	M	$C=M\sqrt{2g}$	Q per foot.
0.5	0.454	3.64	1.28
1.0	0.476	3.82	3.82
1.5	0.478	3.83	7.17
2.0	0.4596	3.688	10.42
2.5	0.4474	3.590	14.19
3.0	0.4425	3.551	18.45
3.5	0.4424	3.550	23.24
4.0	0.4424	3.550	28.50
4.5	0.4424	3.550	33.87
5.0	0.4424	3.550	39.68
5.5	0.4424	3.550	45.79
6.0	0.4424	3.550	52.16



CORNELL UNIVERSITY

EXPERIMENT No. 8.

BAZIN'S SERIES No. 182.

MAY 27TH, 1899.

Length of crest 6.58 ft.

From full line coefficient curve below

Number of experiments, 5.

Limiting heads, 1.695 ft. and 4.745 ft.

Head, in feet. H	M	$C=M\sqrt{2g}$	Q per ft.
0.5	0.5255	4.214	1.50
1.0	0.5283	4.241	4.24
1.5	0.5096	4.088	7.52
2.0	0.4949	3.970	11.22
2.5	0.4848	3.890	15.37
3.0	0.4773	3.830	19.90
3.5	0.4708	3.778	24.75
4.0	0.4661	3.736	29.92
4.5	0.4620	3.708	35.40
5.0	0.4585	3.680	41.14
5.5	0.4548	3.650	47.08
6.0	0.4525	3.630	53.35

CORNELL UNIVERSITY

EXPERIMENT No. 8.

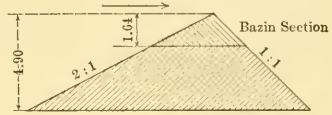
BAZIN'S SERIES No. 182.

MAY 27TH, 1899.

Length of crest 6.58 ft.

Coefficients from dotted curve on coefficient diagram showing effect of a change of position of the mean coefficient curve.

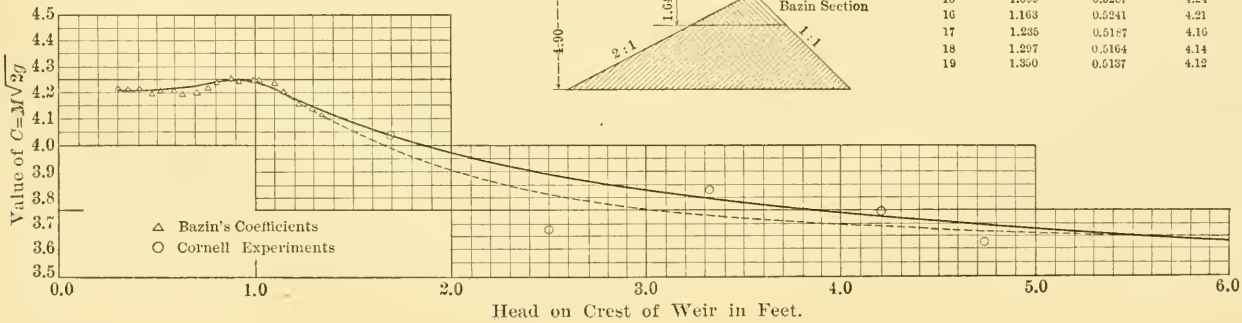
Head, in feet. H	M	$C=M\sqrt{2g}$	Q per ft.
0.5	0.525	4.21	1.50
1.0	0.529	4.24	4.24
1.5	0.505	4.05	7.43
2.0	0.486	3.90	11.02
2.5	0.476	3.82	15.10
3.0	0.467	3.75	19.48
3.5	0.464	3.72	24.35
4.0	0.461	3.70	29.60
4.5	0.459	3.68	35.11
5.0	0.453	3.67	41.03
5.5	0.455	3.65	47.08
6.0	0.453	3.65	53.35



BAZIN'S

SERIES No. 182.

No. of experiment.	h = observed depth on crest, in feet.	m = coefficient of discharge.	$m\sqrt{2g}$
1	0.302	0.5262	4.22
2	0.360	0.5254	4.22
3	0.413	0.5257	4.22
4	0.472	0.5228	4.20
5	0.529	0.5246	4.21
6	0.586	0.5250	4.21
7	0.644	0.5233	4.20
8	0.697	0.5228	4.20
9	0.755	0.5253	4.22
10	0.810	0.5280	4.24
11	0.866	0.5310	4.26
12	0.923	0.5306	4.25
13	0.982	0.5305	4.25
14	1.037	0.5301	4.25
15	1.099	0.5287	4.24
16	1.163	0.5241	4.21
17	1.235	0.5187	4.16
18	1.297	0.5164	4.14
19	1.350	0.5137	4.12



CORNELL UNIVERSITY

EXPERIMENT No. 8.

BAZIN'S SERIES No. 178.

MAY 30TH, 1899.

Length of crest 6.58 ft.

Upstream face covered with $\frac{1}{4}$" mesh galvanized wire screening.

Number of experiments, 5.

Limiting heads, 1.657 ft. and 5.011 ft.

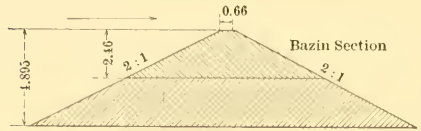
Head, in feet. H	M	$C=M\sqrt{2g}$	Q per ft.
0.5	0.391	3.140 *	1.11
1.0	0.426	3.422	3.42
1.5	0.442	3.548	6.51
2.0	0.450	3.612	10.21
2.5	0.453	3.640	14.39
3.0	0.456	3.658	18.99
3.5	0.457	3.668	24.00
4.0	0.456	3.662	29.30
4.5	0.455	3.653	34.85
5.0	0.454	3.644	40.74
5.5	0.453	3.636	46.88
6.0	0.452	3.630	53.36

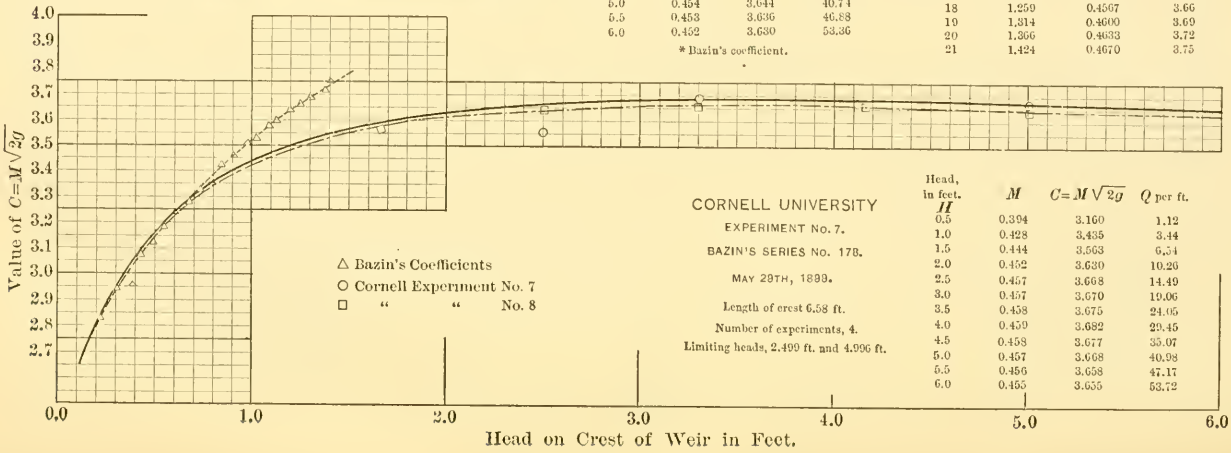
* Bazin's coefficient.

BAZIN'S

SERIES NO. 178.

No. of experiment.	h = observed depth on crest, in feet.	m = coefficient of discharge.	$m\sqrt{2g}$
1	0.222	0.3522	2.83
2	0.299	0.3673	2.95
3	0.367	0.3686	2.96
4	0.431	0.3837	3.08
5	0.491	0.3907	3.13
6	0.556	0.3981	3.19
7	0.614	0.4038	3.24
8	0.669	0.4093	3.28
9	0.732	0.4147	3.33
10	0.789	0.4190	3.36
11	0.847	0.4271	3.43
12	0.906	0.4315	3.46
13	0.966	0.4379	3.51
14	1.028	0.4398	3.53
15	1.083	0.4467	3.58
16	1.142	0.4493	3.60
17	1.195	0.4540	3.64
18	1.259	0.4567	3.66
19	1.314	0.4600	3.69
20	1.366	0.4633	3.72
21	1.424	0.4670	3.75





CORNELL UNIVERSITY

EXPERIMENT No. 7.

BAZIN'S SERIES No. 178.

MAY 29TH, 1899.

Length of crest 6.58 ft.

Number of experiments, 4.

Limiting heads, 2.499 ft. and 4.996 ft.

Head, in feet. H	M	$C=M\sqrt{2g}$	Q per ft.
0.5	0.394	3.160	1.12
1.0	0.428	3.435	3.44
1.5	0.444	3.563	6.54
2.0	0.452	3.630	10.26
2.5	0.457	3.668	14.49
3.0	0.457	3.670	19.06
3.5	0.458	3.675	24.05
4.0	0.459	3.682	29.45
4.5	0.458	3.677	35.07
5.0	0.457	3.668	40.98
5.5	0.456	3.658	47.17
6.0	0.455	3.655	53.72

CORNELL UNIVERSITY

EXPERIMENT NO. 9.

BAZIN'S SERIES NO. 173.

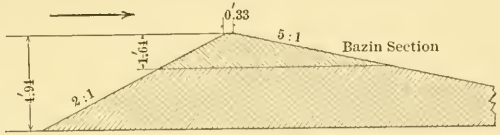
MAY 31ST, 1899.

Length of crest 6.58 ft.

Number of experiments, 4.

Limiting heads, 1.677 ft. and 4.842 ft.

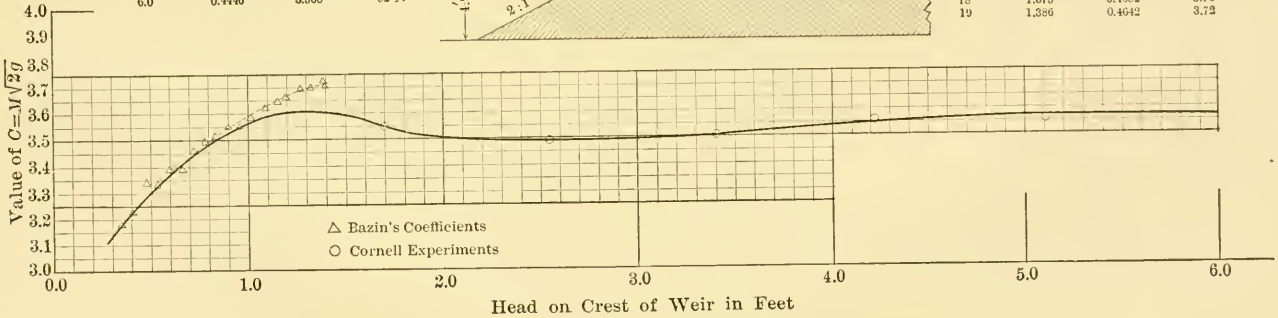
Head, in feet. H	M	$C = M\sqrt{2g}$	Q per foot.
0.5	0.4112	3.300	1.17
1.0	0.4450	3.570	3.57
1.5	0.4480	3.595	6.60
2.0	0.4374	3.510	9.92
2.5	0.4355	3.495	13.83
3.0	0.4356	3.496	18.16
3.5	0.4375	3.510	22.97
4.0	0.4410	3.537	28.30
4.5	0.4436	3.560	33.96
5.0	0.4446	3.568	39.87
5.5	0.4446	3.568	46.01
6.0	0.4446	3.568	52 44



BAZIN'S

SERIES NO. 173.

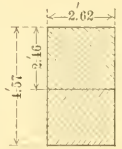
No. of experiment.	h = observed depth on crest, in feet.	m = coefficient of discharge.	$m\sqrt{2g}$
1	0.346	0.3962	3.18
2	0.410	0.4030	3.23
3	0.471	0.4159	3.34
4	0.532	0.4155	3.33
5	0.594	0.4223	3.39
6	0.658	0.4233	3.39
7	0.714	0.4319	3.46
8	0.769	0.4363	3.50
9	0.834	0.4398	3.52
10	0.892	0.4438	3.56
11	0.960	0.4447	3.56
12	1.021	0.4480	3.59
13	1.074	0.4532	3.63
14	1.137	0.4555	3.65
15	1.196	0.4572	3.67
16	1.260	0.4610	3.70
17	1.315	0.4619	3.71
18	1.375	0.4652	3.73
19	1.386	0.4642	3.72



BAZIN'S

SERIES NO. 114.

No. of experiment.	h = observed depth on crest, in feet.	m = coefficient of discharge.	$m\sqrt{2g}$
1	0.204	0.3073	2.47
2	0.280	0.3167	2.54
3	0.352	0.3229	2.59
4	0.433	0.3245	2.60
5	0.504	0.3233	2.59
6	0.578	0.3241	2.60
7	0.657	0.3270	2.62
8	0.735	0.3272	2.63
9	0.810	0.3281	2.63
10	0.882	0.3300	2.65
11	0.958	0.3316	2.66
12	1.034	0.3336	2.68
13	1.112	0.3351	2.69
14	1.171	0.3361	2.70
15	1.243	0.3398	2.73
16	1.301	0.3407	2.73
17	1.384	0.3440	2.76



Bazin's Section

CORNELL UNIVERSITY

EXPERIMENT NO. 10. JUNE 1ST, 1899

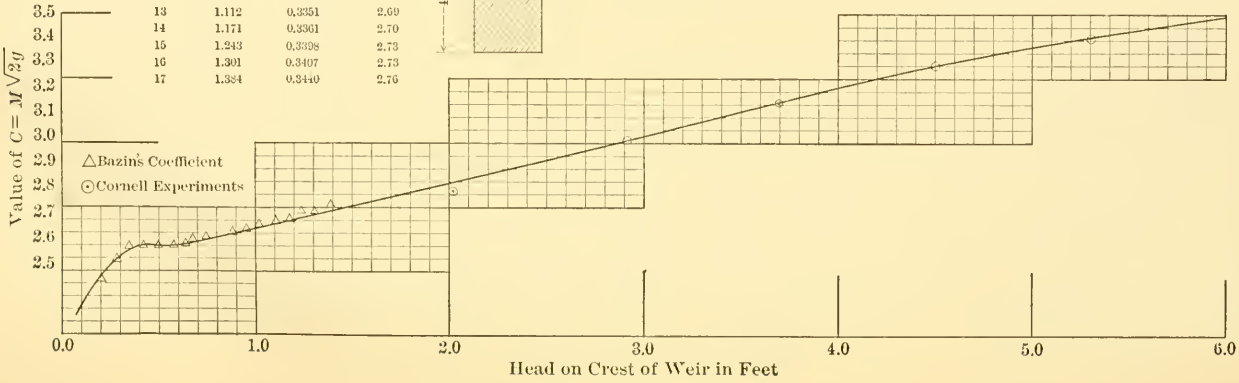
Length of crest 6.58 ft.

Number of experiments, 5.

Limiting heads, 2.038 ft. and 5.324 ft.

Head. In feet. H (1)	M (2)	$C=M\sqrt{2g}$ (3)	Q per ft. (4)
0.5	0.324	2.500 *	0.93
1.0	0.333	2.672 *	2.67
1.5	0.343	2.750 *	5.04
2.0	0.354	2.840	8.02
2.5	0.365	2.928	11.53
3.0	0.376	3.015	15.66
3.5	0.388	3.113	20.37
4.0	0.400	3.208	25.67
4.5	0.411	3.297	31.45
5.0	0.422	3.383	37.86
5.5	0.426	3.420	44.10
6.0	0.433	3.480	51.15

* Bazin's coefficients.



BAZIN'S

SERIES NO. 116.

CORNELL UNIVERSITY

EXPERIMENT NO. 11.

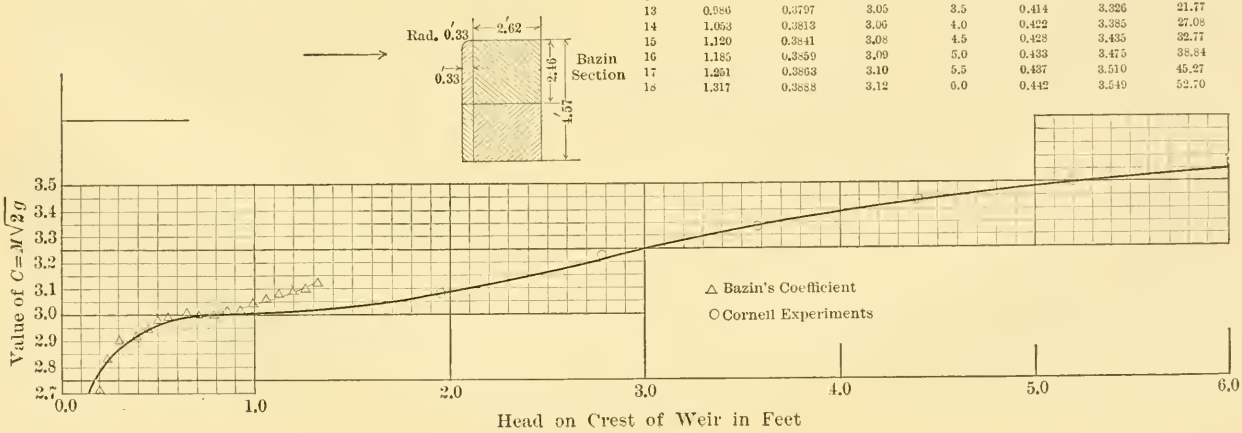
BAZIN'S SERIES NO. 116.

JUNE 1ST, 1899.

Length of crest 6.58 ft.
Number of experiments, 5.
Limiting heads, 1.949 ft. and 5.203 ft.

No. of experiment.	h = observed depth on crest, in feet.	m = coefficient of discharge.	$m\sqrt{2g}$
1	0.177	0.3372	2.71
2	0.225	0.3526	2.83
3	0.296	0.3607	2.90
4	0.367	0.3637	2.92
5	0.435	0.3679	2.95
6	0.504	0.3707	2.98
7	0.537	0.3731	2.99
8	0.639	0.3753	3.01
9	0.713	0.3734	3.00
10	0.781	0.3742	3.00
11	0.849	0.3764	3.02
12	0.917	0.3765	3.02
13	0.986	0.3797	3.05
14	1.053	0.3813	3.06
15	1.120	0.3841	3.08
16	1.185	0.3859	3.09
17	1.251	0.3863	3.10
18	1.317	0.3888	3.12

Head, in feet. H	M	$C = M\sqrt{2g}$	Q per foot.
0.5	0.369	2.960	1.06
1.0	0.375	3.008	3.01
1.5	0.378	3.033	5.57
2.0	0.384	3.085	8.72
2.5	0.388	3.115	12.31
3.0	0.405	3.252	16.89
3.5	0.414	3.326	21.77
4.0	0.422	3.385	27.08
4.5	0.428	3.435	32.77
5.0	0.433	3.475	38.84
5.5	0.437	3.510	45.27
6.0	0.442	3.549	52.70



CORNELL UNIVERSITY

EXPERIMENT NO. 12.

BAZIN'S SERIES NO. 115.

JUNE 20, 1899.

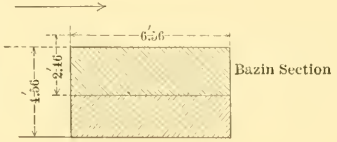
Length of crest 6.58 ft.

Number of experiments, 4.

Limiting heads, 2.175 ft. and 5.102 ft.

Head, in feet. H	M	$C = M\sqrt{2g}$	Q per foot.
0.5	0.3117	2.502 *	0.99
1.0	0.3238	2.600	2.60
1.5	0.3170	2.545	4.67
2.0	0.3095	2.485	7.04
2.5	0.3088	2.480	9.80
3.0	0.3133	2.515	13.06
3.5	0.3193	2.563	16.78
4.0	0.3251	2.610	20.88
4.5	0.3307	2.655	25.35
5.0	0.3364	2.700	30.18
5.5	0.3414	2.740	35.33
6.0	0.3437	2.760	40.55

* Bazin's coefficient.

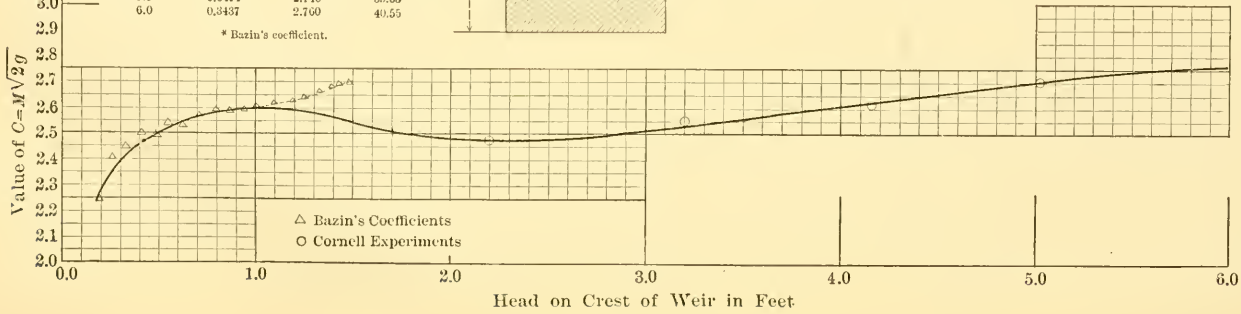


Bazin Section

BAZIN'S

SERIES NO. 115.

No. of experiment.	h = observed depth on crest, in feet.	m = coefficient of discharge.	$m\sqrt{2g}$
1	0.196	0.2747	2.25
2	0.264	0.3004	2.41
3	0.342	0.3047	2.45
4	0.415	0.3123	2.51
5	0.495	0.3119	2.50
6	0.566	0.3184	2.55
7	0.638	0.3173	2.54
8	0.716	0.3194	2.56
9	0.792	0.3243	2.60
10	0.871	0.3245	2.60
11	0.948	0.3245	2.60
12	1.023	0.3250	2.61
13	1.097	0.3281	2.63
14	1.178	0.3294	2.64
15	1.260	0.3302	2.65
16	1.330	0.3342	2.68
17	1.388	0.3348	2.69
18	1.424	0.3365	2.70
19	1.467	0.3362	2.70



CORNELL UNIVERSITY

EXPERIMENT NO. 13

BAZIN'S SERIES NO. 117

JUNE 20, 1899.

Length of crest 6.58 ft.

Number of experiments, 5.

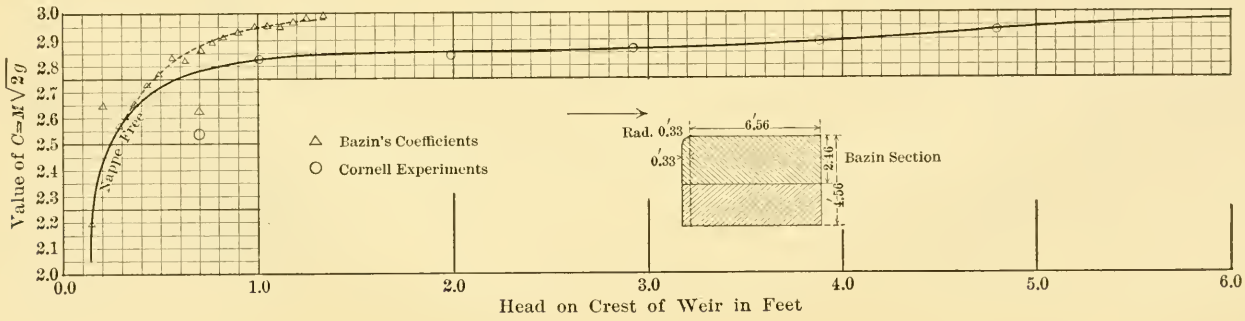
Limiting heads, 1.016 ft. and 4.830 ft.

Head, in feet. H	M	$C=M\sqrt{2g}$	Q per foot.
0.5	0.338	2.716	0.98
1.0	0.352	2.828	2.83
1.5	0.354	2.842	5.22
2.0	0.354	2.844	8.05
2.5	0.355	2.850	11.26
3.0	0.356	2.860	14.85
3.5	0.358	2.877	18.83
4.0	0.360	2.896	23.17
4.5	0.363	2.918	27.84
5.0	0.367	2.945	32.91
5.5	0.369	2.963	38.22
6.0	0.370	2.975	43.70

BAZIN'S

SERIES NO. 117

No. of experiment.	h = observed depth on crest, in feet.	m = coefficient of discharge.	$m\sqrt{2g}$
1	0.158	0.2741	2.19
2	0.204	0.3287	2.64
3	0.289	0.3207	2.57
4	0.361	0.3286	2.65
5	0.426	0.3396	2.73
6	0.494	0.3452	2.77
7	0.562	0.3533	2.83
8	0.635	0.3521	2.82
9	0.708	0.3563	2.86
10	0.771	0.3604	2.89
11	0.834	0.3629	2.91
12	0.912	0.3649	2.93
13	0.989	0.3671	2.95
14	1.064	0.3680	2.95
15	1.129	0.3678	2.95
16	1.197	0.3701	2.97
17	1.267	0.3712	2.98
18	1.336	0.3731	2.99



CORNELL UNIVERSITY

EXPERIMENT NO. 14.

REXFORD FLATS DAM.

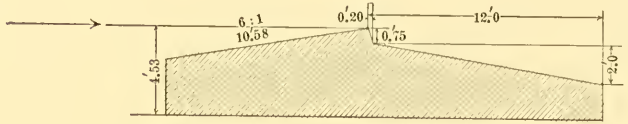
JUNE 3RD, 1899.

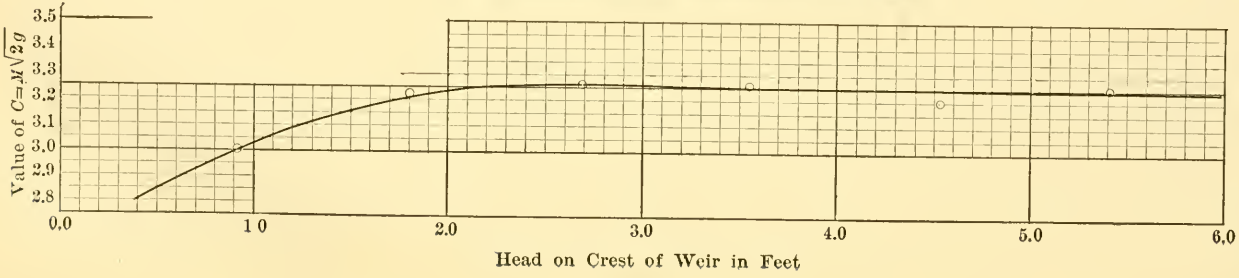
Length of crest 6.58 ft.

Number of experiments, 6.

Limiting heads, 0.924 ft. and 5.418 ft.

Head, in feet. H	M	$C=M\sqrt{2g}$	Q per foot.
0.5	0.355	2.850	1.02
1.0	0.377	3.025	3.02
1.5	0.394	3.160	5.80
2.0	0.403	3.232	9.15
2.5	0.406	3.255	12.86
3.0	0.406	3.255	16.90
3.5	0.405	3.250	21.27
4.0	0.405	3.250	26.00
4.5	0.404	3.245	30.97
5.0	0.404	3.242	36.22
5.5	0.403	3.238	41.76
6.0	0.403	3.236	47.56





CORNELL UNIVERSITY

EXPERIMENT NO. 15.

REXFORD FLATS DAM.

With up-stream edge of crest rounded to a radius of 0.328 ft.

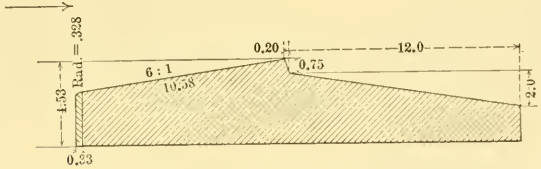
JUNE 30, 1899.

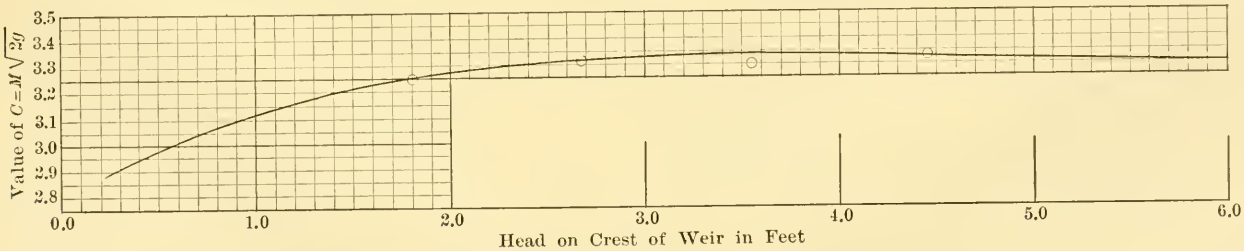
Length of crest 6.58 ft.

Number of experiments, 4.

Limiting heads, 1.786 ft. and 4.439 ft.

Head, in feet. H	M	$C = M\sqrt{2g}$	Q per ft.
0.5	0.371	2.980	1.06
1.0	0.389	3.120	3.12
1.5	0.400	3.210	5.90
2.0	0.408	3.270	9.25
2.5	0.412	3.306	13.07
3.0	0.415	3.332	17.32
3.5	0.416	3.340	21.86
4.0	0.415	3.331	26.65
4.5	0.414	3.325	31.72
5.0	0.413	3.315	37.15
5.5	0.412	3.305	42.65
6.0	0.411	3.300	48.48





CORNELL UNIVERSITY

EXPERIMENT No. 18.

LITTLE FALLS DAM, SECTION No. 1.

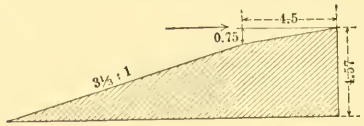
JUNE 5TH, 1899.

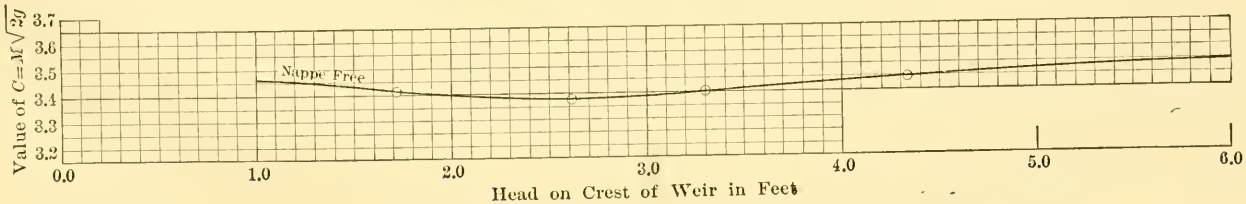
Length of crest 6.58 ft.

Number of experiments, 4.

Limiting heads, 1.735 ft. and 4.339 ft.

Head, in feet. H	M	$C=M\sqrt{2g}$	Q per ft.
0.5			
1.0	0.432	3.468	3.47
1.5	0.428	3.434	6.30
2.0	0.424	3.400	9.62
2.5	0.421	3.380	13.36
3.0	0.422	3.385	17.58
3.5	0.425	3.411	22.32
4.0	0.428	3.436	27.48
4.5	0.431	3.460	33.01
5.0	0.433	3.478	38.87
5.5	0.435	3.490	45.00
6.0	0.436	3.500	51.44





CORNELL UNIVERSITY

EXPERIMENT NO. 17

LITTLE FALLS DAM, SECTION NO. 2.

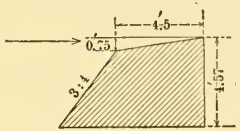
JUNE 5TH, 1899.

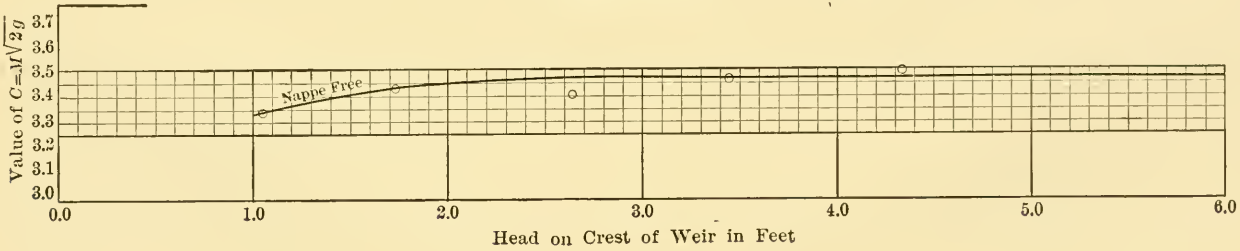
Length of crest 6.58 ft.

Number of experiments, 5.

Limiting heads, 1.067 ft. and 4.336 ft.

Head, in feet. H	M	$C=M\sqrt{2g}$	Q per foot.
0.5			
1.0	0.415	3.330	3.33
1.5	0.425	3.403	6.25
2.0	0.430	3.450	9.76
2.5	0.432	3.465	13.70
3.0	0.432	3.470	18.03
3.5	0.432	3.465	22.66
4.0	0.432	3.465	27.75
4.5	0.432	3.465	33.05
5.0	0.432	3.465	38.75
5.5	0.432	3.465	44.68
6.0	0.432	3.465	50.90





CORNELL UNIVERSITY

EXPERIMENT NO. 18.

SECTION OF SPILLWAY OF INDIAN LAKE DAM.

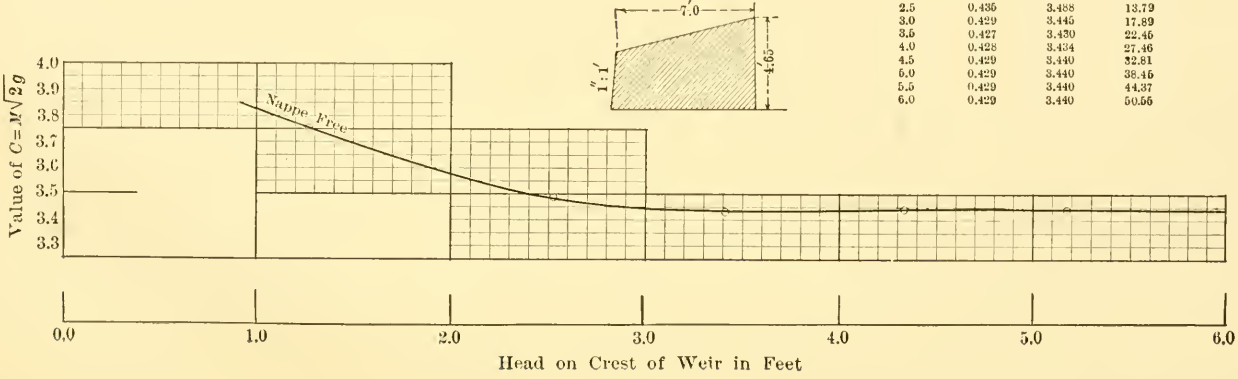
JUNE 6TH, 1899.

Length of crest 6.58 ft.

Number of experiments, 5.

Limiting heads, 1.277 ft. and 5.180 ft.

Head, in feet. H	M	$C=M\sqrt{2g}$	Q per foot.
0.5			
1.0	0.476	3.820	3.82
1.5	0.466	3.695	6.78
2.0	0.458	3.675	10.38
2.5	0.436	3.488	13.79
3.0	0.429	3.445	17.89
3.5	0.427	3.430	22.46
4.0	0.428	3.434	27.46
4.5	0.429	3.440	32.81
5.0	0.429	3.440	38.46
5.5	0.429	3.440	44.37
6.0	0.429	3.440	50.56



CORNELL UNIVERSITY

EXPERIMENT NO. 19.

SECTION FOR SUBMERGED WEIR.

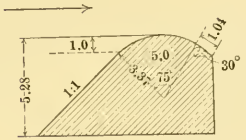
JUNE 7TH, 1899.

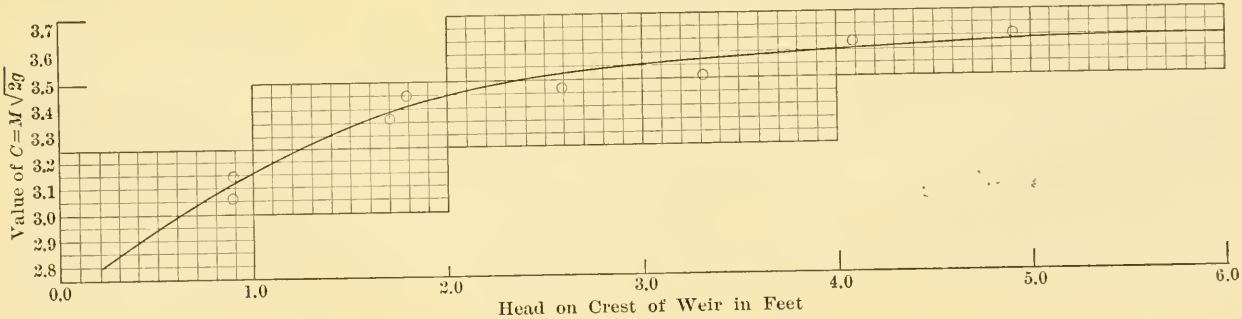
Length of crest 6.58 ft.

Number of experiments, 8

Limiting heads, 0.887 and 4.913 ft.

Head, in feet. H	M	$C=M\sqrt{2g}$	Q per ft.
0.5	0.367	2.945	1.04
1.0	0.394	3.165	3.17
1.5	0.412	3.305	6.06
2.0	0.430	3.454	9.77
2.5	0.439	3.520	13.91
3.0	0.443	3.558	18.48
3.5	0.447	3.585	23.46
4.0	0.450	3.610	28.88
4.5	0.452	3.630	34.63
5.0	0.455	3.650	40.80
5.5	0.457	3.665	47.28
6.0	0.458	3.675	54.02





On April 14th, 1899, the writer visited the new Cornell University Hydraulic Laboratory and at once saw that a fine opportunity was offered there to experiment on flows over weirs at much higher heads than had hitherto been possible; and, on communicating his views to the United States Board of Engineers on Deep Waterways, he was permitted to undertake a series of experiments in co-operation with the University authorities. Messrs. Wallace Greenalch, Assoc. M. Am. Soc. C. E., Robert E. Horton, and George E. Cook were detailed from the Deep Waterways engineering corps for this work, which was done in co-operation with Professor Gardner S. Williams, M. Am. Soc. C. E., Engineer in Charge of the Hydraulic Laboratory, permission to use the same having been obtained by correspondence with Professor E. A. Fuertes, M. Am. Soc. C. E., Director of Cornell University College of Civil Engineering. The writer gave the experiments general supervision, the working out of the details being mostly left to Mr. Greenalch and Professor Williams, Mr. Greenalch undertaking to construct the necessary flumes, bulkheads, experimental weirs, etc., and Professor Williams preparing and taking charge of the measuring apparatus. The reductions were made by Messrs. Greenalch, Horton and Cook under the direction of the writer.

Fig. 1 is a plan and section of the experimental channel at the Cornell University Hydraulic Laboratory. This laboratory has been quite fully described in *Engineering News* for March 2d, 1899, and no farther description will be given here than is necessary to explain the experiments.

Description of the Arrangements for the Experiments.—The canal in which the experiments were made consists, briefly, of a channel with sides and bottom of concrete. It is 418 ft. long, 16 ft. wide and 10 ft. deep. The gradient of the bottom of the channel is at the rate of 1 ft. in 500. A bulkhead composed of 12 x 12-in. timbers, situated about 60 ft. from the upper end, divides the channel into two chambers. A standard sharp-edged weir, 16 ft. in length, was placed on this bulkhead, the crest of the weir being 13.13 ft. above the bottom of the channel. The upper chamber above the bulkhead has higher side walls than the lower chamber, which admitted of a depth of 17.7 ft. of water. At the lower end of the channel another timber bulkhead closes the lower chamber, and on this the weirs to be experimented upon were built. The top of this bulkhead was about 4.8 ft. above the

bottom of the channel. The heights vary slightly for each experimental weir, the exact height of each being shown on the sections at the head of the tabulations of results.

In order to obtain heads of about 5 ft. on the lower weir the 16-ft. channel was narrowed to a width of 6.56 ft. (2 m.) by means of a wooden flume, as shown in Fig. 1. This flume was 6.56 ft. wide for a distance of 48 ft. above the lower bulkhead and then expanded to a width of 16 ft. in a length of 8.3 ft., as shown. The flume was constructed of matched, white pine boards 1.75 ins. thick, planed on the inside and held in place by bents of 4 x 4-in. timbers. As the lower bulkhead was water-tight for the whole width of the channel, no attempt was made to construct the sides and bottoms of the flume absolutely water-tight, although they were practically so. Inasmuch as nearly equal pressure on both sides of flume would permit of greater economy in construction, two boards were left off each side at the upper end, thus allowing the water to enter at the sides between the flume and the concrete walls of the main canal. This arrangement also diminished greatly the area of water-tight work. The sides of the flume were extended from 8 to 24 ft. below the bulkhead, according to the form of weir experimented upon, thus preventing lateral expansion of the nappe after passing the weir and crest. Openings were left in each side of this extension below the level of the crest in order to certainly allow free access of air under the nappe. The vertical fall of the water from the crest of the experimental weir to the rock below was about 12.2 ft.

There is a pond of 22 acres above the main reservoir dam, the surface of which was raised about 1.7 ft. by means of flash boards placed on the main spillway. The water surface thus obtained was about 5.4 ft. above the crest of the standard weir. Water from the reservoir was admitted into the upper chamber through six wooden sluice gates, operated by rack and pinion apparatus with long levers.

The sharp-edged standard weir was composed of a 3.5 x 5-in. steel angle, secured by lag screws to a 6 x 12-in. oak timber, as shown in Fig. 1. After bolting the angle-iron into place on the timber, the edge of the 5-in. leg was planed and dressed to a true line $\frac{5}{16}$ in. in width, and carefully leveled in position on the upper bulkhead. Air was admitted freely under the nappe by means of deflecting boards at each end of the weir. The fall of the water from the crest of the standard

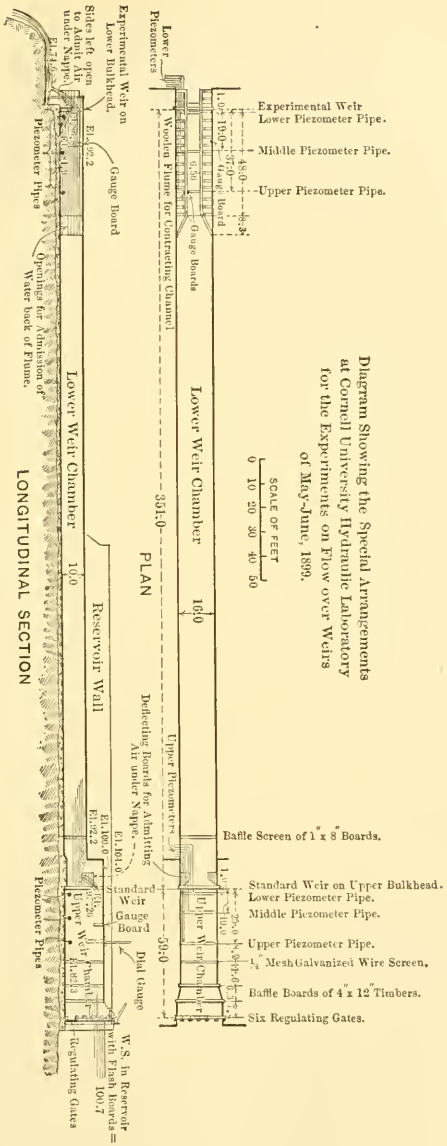


Diagram Showing the Special Arrangements at Cornell University Hydraulic Laboratory for the Experiments on Flow over Weirs of May-June, 1899.
SCALE OF FEET
0 10 20 30 40 50
Experimental Weir Lower Piezometer Pipe.
Middle Piezometer Pipe.
Upper Piezometer Pipe.
Lower Piezometers
Woolen Plume for Contracting Channel
Gauge Boards
Lower Weir Chamber
160
Upper Piezometers
Baffle Screen of 1" x 8" Boards.
Standard Weir on Upper Bulkhead.
Lower Piezometer Pipe.
Middle Piezometer Pipe.
Upper Piezometer Pipe.
¼" Mesh Galvanized Wire Screen.
Baffle Boards of 4" x 12" Timbers.
Six Regulating Gates.
Experimental Weir on Lower Bulkhead.
Sides left open to Admit Air under Nappe
Gauge Board
Openings for Admission of Water back of Plume.
Piezometer Pipes
Reservoir Wall
Deflecting Boards for Admitting Air under Nappe
Standard Weir
Gauge Board
Dial Gauge
W.S. in Reservoir with Flash Boards = 100.7
Regulating Gates
PLAN
LONGITUDINAL SECTION

FIG. 1.

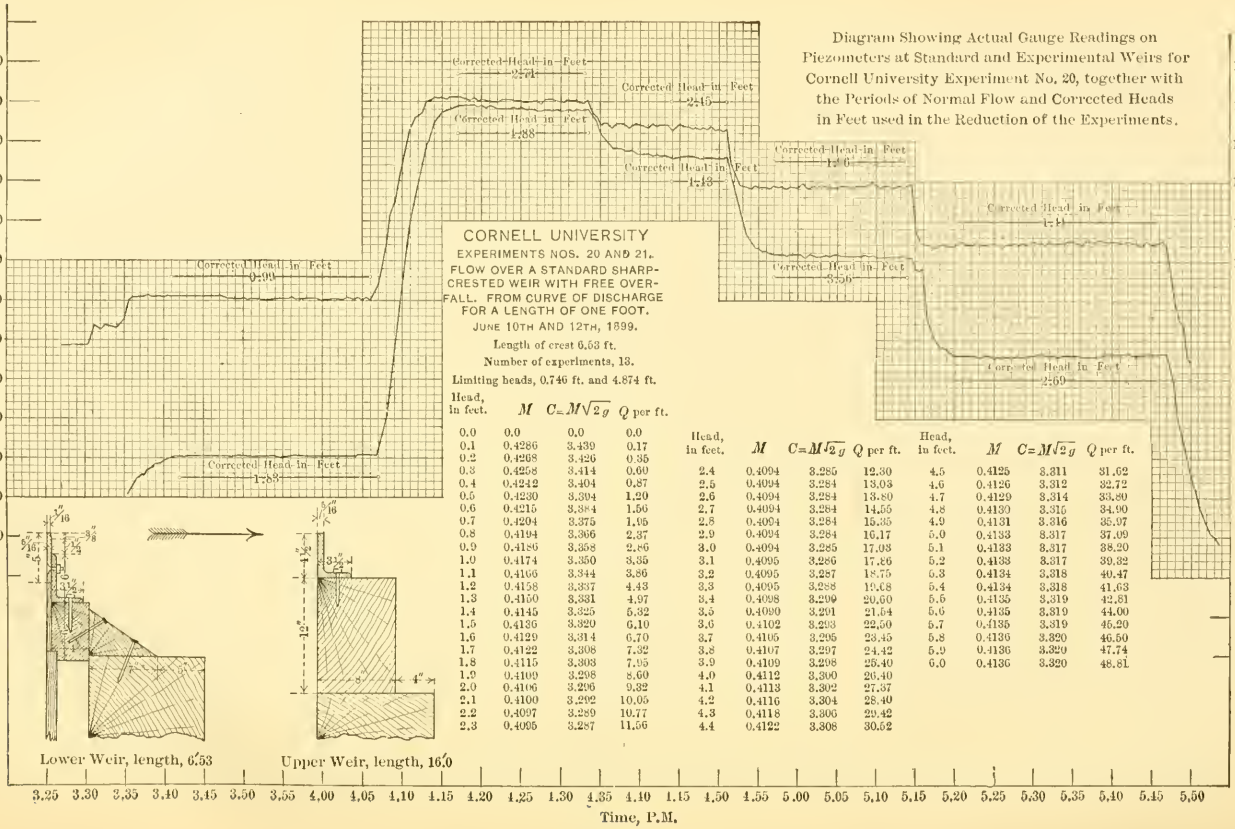
weir to the water surface in the lower chamber varied from about 3 to 8 ft., according to the quantity of water flowing.

The velocity of the water passing through the regulating gates at the extreme head of the channel was checked by three screens in the upper chamber. The first two consisted of 4 x 12-in. timbers placed horizontally with the wide face toward the current, and spaced from 8 to 12 ins. apart. Below these was a third screen of $\frac{1}{4}$ in. mesh, galvanized wire netting. A screen, composed of 1 x 8 in. boards, laid horizontally with the edges to the current and spaced 2 ins. apart, was placed about 20 ft. below the upper bulkhead, and served to quiet the water in the lower chamber.

The heads on the weirs were measured by means of piezometers, constructed as follows: A 1-in. galvanized iron pipe, with holes $\frac{1}{4}$ in. in diameter and spaced 6 ins. apart, was laid across the channel about 8 ins. above the bottom, with the holes therein opening downward. Connections with these pipes were made by $\frac{3}{4}$-in. pipes passing through the bulkhead to a point below the weir, where the gauges could easily be connected by rubber hose. The gauges were glass tubes, $\frac{3}{4}$ in. internal diameter, mounted on wooden standards, and read by a scale graduated to 2-mm. spaces. Three piezometers were set at each weir, as shown in Fig. 1, though readings were taken only on the upper two. In order to check the accuracy of the piezometric readings, at the conclusion of Experiment No. 17, a fourth piezometer pipe was set in the bottom of the flume above the lower bulkhead, and about 6 ins. up stream from the upper piezometer. This pipe was set with $\frac{1}{4}$-in. holes directly on top, and with the top of the pipe flush with the bottom of the flume. Readings were taken simultaneously on both piezometers, and considerable differences noted.

At the lower weir the height of the flowing water in the flume and of the still water behind the flume was read on scales marked on the side of the flume. These scales were divided to 0.05 ft. and read by interpolation to 0.01 ft. Similar gauges were set in the upper chamber and in the reservoir, and readings of each were taken every five minutes.

In addition to the gauge board in the upper chamber, a float gauge, read by dial to 0.01 ft., was set directly over the upper piezometer. The float of this dial gauge was a heavy, sheet-tin, air-tight vessel, weighted with shot and caused to move vertically with the water in the interior of a length of 8-in. cast-iron pipe, suspended from two timbers



CORNELL UNIVERSITY

EXPERIMENTS NOS. 20 AND 21.

FLOW OVER A STANDARD SHARP-CRESTED WEIR WITH FREE OVERFALL. FROM CURVE OF DISCHARGE FOR A LENGTH OF ONE FOOT.

JUNE 10TH AND 12TH, 1899.

Length of crest 6.53 ft.

Number of experiments, 13.

Limiting heads, 0.746 ft. and 4.874 ft.

Head, in feet.	M	$C=M\sqrt{2g}$	Q per ft.
0.0	0.0	0.0	0.0
0.1	0.4286	3.439	0.17
0.2	0.4268	3.426	0.35
0.3	0.4258	3.414	0.60
0.4	0.4242	3.404	0.87
0.5	0.4230	3.394	1.20
0.6	0.4216	3.384	1.56
0.7	0.4204	3.375	1.95
0.8	0.4194	3.366	2.37
0.9	0.4186	3.358	2.86
1.0	0.4174	3.350	3.35
1.1	0.4166	3.344	3.86
1.2	0.4158	3.337	4.43
1.3	0.4150	3.331	4.97
1.4	0.4145	3.325	5.32
1.5	0.4136	3.320	6.10
1.6	0.4129	3.314	6.70
1.7	0.4122	3.308	7.32
1.8	0.4115	3.303	7.95
1.9	0.4109	3.298	8.60
2.0	0.4106	3.296	9.32
2.1	0.4100	3.292	10.05
2.2	0.4097	3.289	10.77
2.3	0.4095	3.287	11.56
2.4	0.4094	3.285	12.30
2.5	0.4094	3.284	13.03
2.6	0.4094	3.284	13.80
2.7	0.4094	3.284	14.55
2.8	0.4094	3.284	15.35
2.9	0.4094	3.284	16.17
3.0	0.4094	3.285	17.03
3.1	0.4095	3.286	17.86
3.2	0.4095	3.287	18.75
3.3	0.4095	3.288	19.68
3.4	0.4098	3.290	20.60
3.5	0.4099	3.291	21.54
3.6	0.4102	3.293	22.50
3.7	0.4105	3.295	23.45
3.8	0.4107	3.297	24.42
3.9	0.4109	3.298	25.40
4.0	0.4112	3.300	26.40
4.1	0.4113	3.302	27.37
4.2	0.4116	3.304	28.40
4.3	0.4118	3.306	29.42
4.4	0.4122	3.308	30.52
4.5	0.4125	3.311	31.62
4.6	0.4126	3.312	32.72
4.7	0.4129	3.314	33.80
4.8	0.4130	3.315	34.90
4.9	0.4131	3.316	35.97
5.0	0.4133	3.317	37.09
5.1	0.4133	3.317	38.20
5.2	0.4133	3.317	39.32
5.3	0.4134	3.318	40.47
5.4	0.4134	3.318	41.63
5.5	0.4135	3.319	42.81
5.6	0.4135	3.319	44.00
5.7	0.4135	3.319	45.20
5.8	0.4136	3.320	46.50
5.9	0.4136	3.320	47.74
6.0	0.4136	3.320	48.81

FIG. 2.

across the upper channel. The dial was placed in such a position as to be read easily by the assistants operating the gates at the upper end of the channel, thus insuring that the depth over the standard weir be easily maintained at substantially a constant head, as required.

The experimental weirs of different sections were all made of planed, white pine boards about 1 in. thick, so fastened together with screws as to be easily removable. Leakage from the bulkheads and gates at the lower end of the channel was effectively stopped by caulking with oakum and pitch, and also by the application of wheat bran from time to time.

Observations showed that the upper bulkhead was practically water-tight, but at the side gates, near the lower bulkhead, there was a slight leakage, which has been taken at 0.5 cu. ft. per second for high heads and 0.25 cu. ft. per second for low heads, and proportionally between. The main channel, with side walls of concrete, is considered to be water-tight.

The foregoing allowances for leakage are taken to cover the slight evaporation and absorption loss into the sides of the main channel.

Reduction of the Experiments.—The method of conducting the experiments was, in general, as follows: The main head gates at the entrance to the canal were open to such a width that the dial gauge showed a head of 3 ft. above the standard 16-ft weir. They were retained in this position until a uniform regimen of flow was established in the canal and maintained for a period of from 10 to 20 minutes, during which time the piezometers were read at both weirs at intervals of 30 seconds. At the close of such a period the head gates were lowered until the dial gauge showed a reading of 2.5 ft. head on the upper weir, sufficient time being allowed to elapse to establish and maintain a new regimen of flow, the same as before. In this way piezometric observations were taken during several periods at different heads in each experiment, usually terminating with a head of about 6 ins. or 1 ft. on the upper weir. In some cases the varying of the head on the upper weir by uniform decrements of 6 ins. was not adhered to.

The method of treating the piezometric readings, obtained as described in the preceding paragraph, is shown in Figs. 2 and 3. In Fig. 2 the upper curve shows the readings taken at the upper weir (16 ft. in length), and the lower curve the readings taken at the lower experimental weir for Experiment No. 14 on the Rexford Flats dam. These

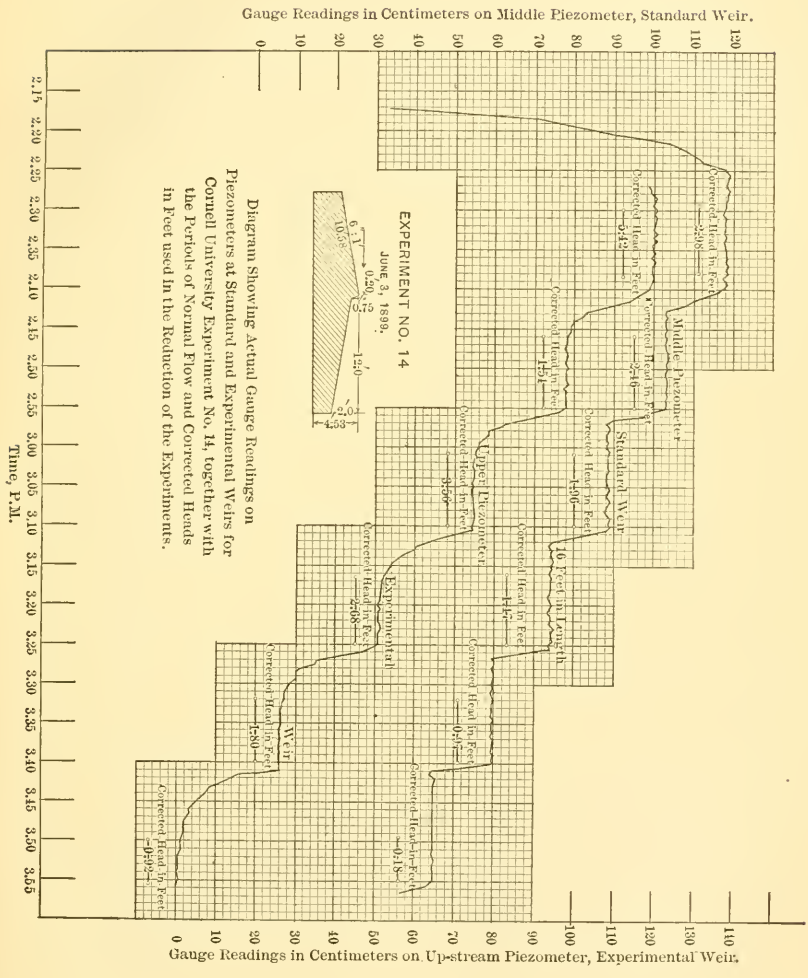


Diagram Showing Actual Gauge Readings on Piezometers at Standard and Experimental Weirs for Cornell University Experiment No. 14, together with the Periods of Normal Flow and Corrected Heads in Feet used in the Reduction of the Experiments.

FIG 8.

two curves, as plotted, show the actual readings taken, in centimeters, without corrections of any sort or kind. The several periods (A-A) of the upper curve, and the corresponding periods (B-B) of the lower curve, represent the actual periods taken in the reductions for each height experimented upon. A mean of the actual readings for these periods has been taken as the mean head during each experimental period.

Fig. 3 is a plotting of similar curves for Experiment No. 20 on a sharp-crested weir, made on June 10th, 1899. The explanations for Fig. 2 apply equally to this figure. Attention may be directed to the method of exhibiting the continuous curve of flow, as shown in Figs. 2 and 3. Its use in the present case is to be credited to Professor Williams. The writer has never seen it used before, and, if it is original with Professor Williams, he is entitled to very great credit for this particular feature of the experiments.

In order to calibrate the upper weir (16 ft. in length), and thereby determine the quantity of water flowing in the lower canal and over the experimental weirs, Experiments Nos. 20 and 21 were made. These experiments apply to a sharp-crested weir of standard form, 5.26 ft. in height, placed in the lower end of the canal. As a basis for the reduction of Experiments Nos. 20 and 21 a discharge curve has been computed for the upper weir for heads up to 0.6 m. (1.969 ft.), using Bazin's formula:

$$Q = M\,L\,H\sqrt{2\,g\,H}$$

$$M = n\left[1 + 0.55\left(\frac{H}{p+h}\right)^2\right]$$

$$m = n\left[1 + 0.55\left(\frac{h}{p+h}\right)^2\right]$$

in which

Q = discharge over weir, in cubic feet per second;
h = observed head on crest, in feet;
h_v = correction for velocity of approach;
H = head on weir corrected for velocity of approach $= h + h_v$;
m = coefficient of discharge in the formula $m\,l\,h\sqrt{2\,g\,h}$;
M = coefficient of discharge in the formula $M\,L\,H\sqrt{2\,g\,H}$;
C = coefficient in the Francis formula $C = M\sqrt{2\,g}$, when g is expressed in feet;
p = height of crest of weir above bottom of channel of approach, in feet;
n = a coefficient which depends on p and h, and which has been taken from Bazin's table.*

* See *Annales des Ponts et Chaussées*, 1888, p. 146.

In the reduction of Experiments Nos. 20 and 21, the mean readings of the middle piezometer at the upper weir were added to the difference of elevation of the gauge zero and of the mean crest as a basis for computing the flow in the upper channel. The discharge corresponding to the heads so obtained was then taken off the discharge curve computed from Bazin's formula for all the periods in which the observed head was less than 0.6 m. (1.969 ft.). The approximate correction to be applied for velocity of approach to the standard weir was then computed as follows :

Let H = the true head on crest of weir, in feet;

h = the observed head, in feet ;

Q_h = discharge over the weir under the head h per lineal foot of crest ;

Q = discharge under the head H per lineal foot of crest ; and

p = height of weir crest above channel bottom, in feet. Then

velocity of approach $= v = \sqrt{2g(H-h)}$,

also,

$$v = \frac{Q}{p+h} = \frac{Q}{A} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

$$\text{and } h_v = (H-h) = \frac{v^2}{2g} = \left(\frac{Q}{A}\right)^2 \times \frac{1}{2g} \quad \ldots\ldots\ldots\ldots (2)$$

Q_h being determined from the discharge curve, an approximate value of v and of the corresponding velocity head was computed and the approximate value of the velocity head so obtained added to the observed head h, which was used in determining Q, v and $(H-h)$ with more precision. Generally, two successive applications of these formulas were found sufficient to determine the velocity head with the desired degree of accuracy. In this way the final corrected head $H = h + h_v$ was obtained. After applying a correction for leakage, percolation and surface evaporation, the corresponding discharges by Bazin's formula have been used in determining the coefficients for the sharp-crested experimental weir (6.53 ft. in length) for heads up to 3.5 ft., as produced by heads not exceeding 2 ft. on the upper weir (16 ft. in length).

The foregoing correction for velocity of approach is merely the addition to the observed heads of $\frac{v^2}{2g}$, as determined for the actual flows of each experiment. Messrs. Fteley and Stearns have experi-

mented on the effect of velocity of approach, especially with reference to that part of it represented by the *vis viva* of the water, and state in their classical paper* that, for the conditions of their experiments, corrections of velocity of approach to be added to the observed heads are best represented by

$$1.45 \text{ to } 1.50 \times \left(\frac{v^2}{2g}\right).$$

The problem of correction for velocity of approach is discussed at length by Hamilton Smith, Jr., M. Am. Soc. C. E., in his "Hydraulics," the conclusion being that, for a weir with full contraction and having an unobstructed channel of considerable length, the correction should be about

$$1.1 \text{ to } 1.25 \times \left(\frac{v^2}{2g}\right).$$

For end contractions suppressed, he adopts the values

$$1.33\left(\frac{v^2}{2g}\right) \text{ and } 1.40\left(\frac{v^2}{2g}\right).$$

In the present case it has seemed preferable to use

$$h_v = \frac{v^2}{2g},$$

although the entire suppression of end contractions might appear to indicate a higher correction for the velocity of approach. This view, however, is based upon other considerations, namely, the actual locations of the piezometers.

Messrs. Fteley and Stearns have pointed out that for standard sharp-crested weirs the head should be measured about 6 ft. back from the crest, but in the present case the heads have been measured much farther back. At the upper 16-ft. weir the heads were measured at the middle piezometer for all experiments, except Nos. 1, 2, 3 and 4, for which they were measured at the upper piezometer. These latter observations have been reduced to the basis of the middle piezometer—which was 10 ft. back from the weir—by methods to be detailed farther on.

At the lower or experimental weirs the heads were measured for all experiments at the upper piezometer 38 ft. above the bulkhead on which the experimental weirs were placed. This location was selected in order to insure the piezometers being well above the long back

Transactions, Am. Soc. C. E., Vol. xii, p. 1.

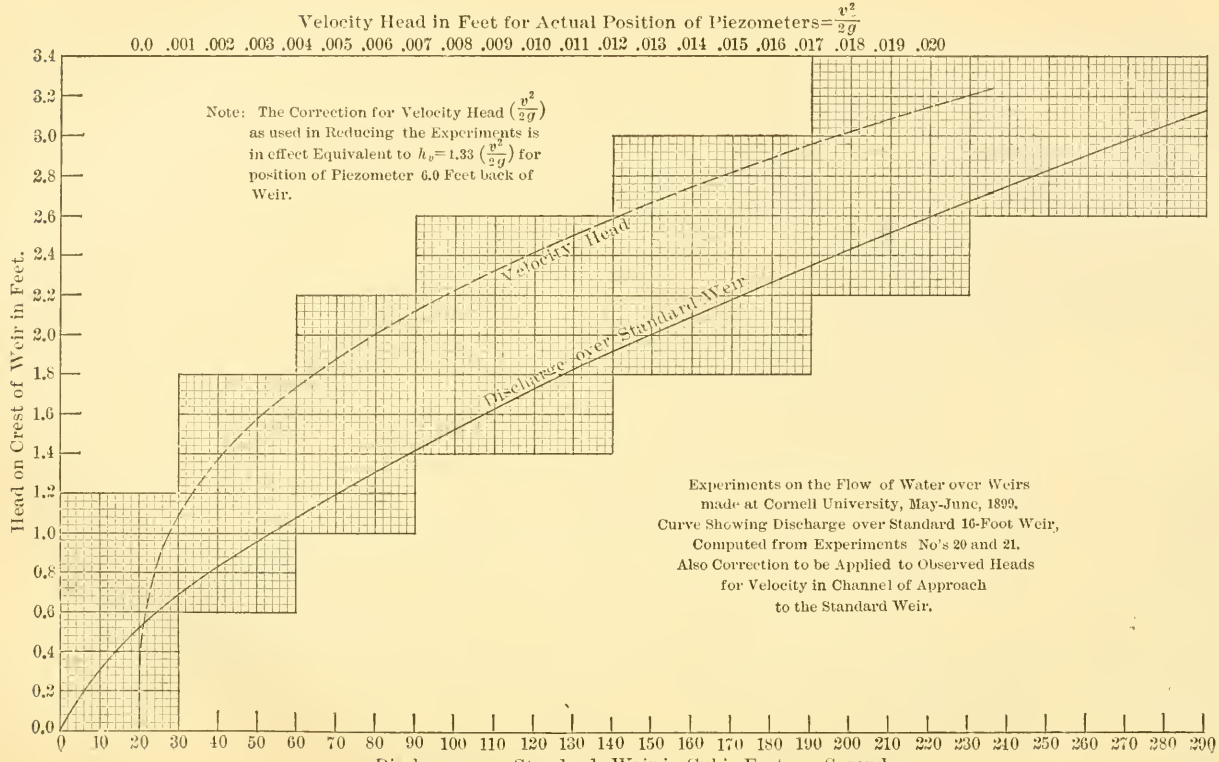

Velocity Head in Feet for Actual Position of Piezometers=$\frac{v^2}{2g}$
0.0 .001 .002 .003 .004 .005 .006 .007 .008 .009 .010 .011 .012 .013 .014 .015 .016 .017 .018 .019 .020
Note: The Correction for Velocity Head $(\frac{v^2}{2g})$ as used in Reducing the Experiments is in effect Equivalent to $h_v = 1.33\ (\frac{v^2}{2g})$ for position of Piezometer 6.0 Feet back of Weir.
Velocity Head
Discharge over Standard Weir
Experiments on the Flow of Water over Weirs made at Cornell University, May-June, 1899.
Curve Showing Discharge over Standard 16-Foot Weir, Computed from Experiments No's 20 and 21.
Also Correction to be Applied to Observed Heads for Velocity in Channel of Approach to the Standard Weir.
Head on Crest of Weir in Feet.
0.0 0.2 0.4 0.6 0.8 1.0 1.2 1.4 1.6 1.8 2.0 2.2 2.4 2.6 2.8 3.0 3.2 3.4
0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 210 220 230 240 250 260 270 280 290
Discharge over Standard Weir in Cubic Feet per Second.

FIG. 4.

slopes of Experiments Nos. 2, 3, 14, 15 and 16, or any other similar series which it might appear desirable to make.

Messrs. Fteley and Stearns remark that the only inaccuracy to come from measuring the heads more than 6 ft. back will be that due to surface slope. We will now examine as to the possible effect of this in the present case.

For an observed head of 2.693 ft. on the standard 16-ft. weir

$$\frac{v^2}{2g} = 0.013 \text{ ft.} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (a)$$

and

$$1.33 \left(\frac{v^2}{2g}\right) = 0.017 \text{ ft.} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (b)$$

The difference of 0.004 ft. is far enough within the limit of accuracy to be negligible.

The corresponding observed head on the experimental weir, 6.56 ft. in length, is 4.677 ft., giving

$$\frac{v^2}{2g} = 0.198 \text{ ft.} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (c)$$

also

$$1.33 \left(\frac{v^2}{2g}\right) = 0.270 \text{ ft.} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (d)$$

The difference is 0 072 ft.

Taking the formula $v = C\sqrt{r\,s}$, in which $r = \frac{A}{P}$ and $s = \frac{h}{l}$, and with l equal to $(38 - 6) = 32$ ft.—the distance between where the piezometer should have been to comply with theoretical conditions for a standard weir and where it was actually set—and computing for h_s under a head of 4.677 ft. on the experimental sharp-crested weir, we find $h_s = 0.061$ ft., which differs from the preceding difference of 0.072 ft. by 0.011 ft. At slightly lower heads this difference disappears so rapidly as to become inappreciable, so far as effect on the coefficients of discharge is concerned.

It was concluded, therefore, that for the conditions of the present case,

$$h_v = \frac{v^2}{2g}$$

gave more nearly the true correction than any other accepted formula.

In his experiments comparing flows over standard weirs with flows over the several experimental sections, Bazin himself did not make

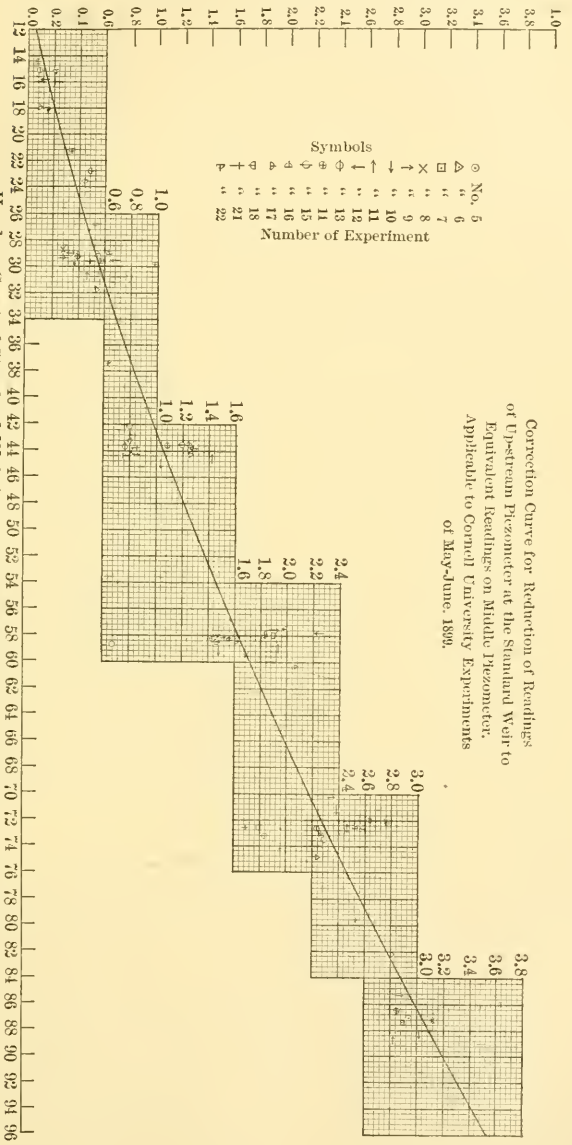

Corrections in Centimeters to be added to Heads from Up-stream Piezometer.
Correction Curve for Reduction of Readings of Up-stream Piezometer at the Standard Weir to Equivalent Readings on Middle Piezometer. Applicable to Cornell University Experiments of May-June, 1899.
Symbols
No. 5 6 7 8 9 10 11 12 13 11 15 16 17 18 21 22
Number of Experiment
Head on Crest of Standard Weir in Centimeters, from Up-stream Piezometer Readings.

Fig. 5.

any corrections for velocity of approach. By working on the ratio $\frac{m}{m'}$ the necessity for such corrections was substantially eliminated. On this point, note what he says on page 259.

Final coefficients of discharge for the several weirs have been obtained from the formula

$$C = M\sqrt{2g} = \frac{Q}{L H^{\frac{3}{2}}}$$

in which

Q = total flow over experimental weir, in cubic feet per second;

L = length of crest of experimental weir, in feet; and

H = final corrected head on experimental weir, in feet. Q having been previously found, H was determined from the mean of the readings, for each period of experiment, of a piezometer connected to a horizontal tube placed flush with the bottom of the channel of approach. The correction for velocity of approach was then computed by the formulas just given, using appropriate values of p and h, Q being known from the previous work, as described.

The values of the coefficients M and C, connected by the relation

$$C = M\sqrt{2g},$$

having been determined for the actual heads deduced from the experiments, the values of C so obtained were plotted and a mean curve drawn to best represent the observations. A new series of coefficients, advancing by equal increments of H, have been read from the curve so obtained, as shown by the tabulations of the experiments, Nos. 1 to 21, on pages 267 to 284, inclusive, and in Fig. 2.

Values of Q, the discharge per lineal foot of crest, have been computed by the formula

$$Q = C H^{\frac{3}{2}}.$$

Having obtained the discharge over the sharp-crested experimental weir for heads up to 3.5 ft., the curve of discharge for the standard weir was extended to the height of 3 ft. by using the coefficients obtained as just described. Fig. 4 shows this curve, as well as the correction curve to be applied to observed heads for velocity in the channel of approach to the standard weir. The additional experiments made on a sharp-crested weir at heads above 3.5 ft. were then reduced in precisely the same manner as before, giving finally a series of coefficients of discharge over a sharp-crested weir with a range of heads from 0.746 ft. to 4.874 ft.

Experiments on the Flow of Water over Weirs
made at Cornell University, May-June, 1899.
Curve Showing Correction to be Added to Readings
of Piezometer placed Eight Inches above Bottom of
Channel of Approach at Experimental Weir to Reduce
to Equivalent Readings on Flush Piezometer.
From Experiments No. 17 and 18, June 6 and 7, 1899.
△ Experiment No. 18, June 6, 1899.
⊙ Experiment No. 19, June 7, 1899.
Difference of Readings of Flush and Upper Piezometers in Centimeters.
0 1 2 3 4 5 6 7 8 9 10
0 5 10 15 20 25 30 35 40 45 50 55 60 65 70 75 80 85 90
Observed Heads in Centimeters on Upper Piezometer at Standard Weir.

FIG. 6.

The method used in reducing the experiments on weirs having crests of irregular profile is the same as for the sharp-crested weir, with the following exceptions:

In addition to the piezometric observations, direct observations of the head on each weir were taken on gauge boards situated above the standard and experimental weirs, respectively. The agreement between the heads so derived and those deduced from the mean piezometric readings is close in all cases, with the exception of the observed heads on the experimental weir in Experiments Nos. 1, 2, 3 and 4. Without going into the detail, it may be stated that for these four experiments, which were the first made, the heads directly observed on the gauge boards are apparently the more reliable, and they have accordingly been used in reducing these four experiments.

Again, in Experiments Nos. 1, 2, 3 and 4, no observations were taken on the middle piezometer at the standard weir. In order to reduce the observed heads as actually taken on the up-stream piezometer at this weir to equivalent heads on the middle piezometer, a correction has been applied, the value of which was obtained as follows: Plotting the difference between the readings of the upper and middle piezometers as ordinates for all experiments in which readings were taken on both, and using the observed heads on the up-stream piezometer as abscissas, a mean curve has been drawn, from which the correction to be applied to any reading on the up-stream piezometer can be read directly. This curve is shown in Fig. 5. The reason for using the readings of the middle, in preference to those taken on the up-stream piezometer at the standard weir, is that the former agree more closely, on the whole, with the readings of the gauge board; also, the middle piezometer, which is only 10 ft. distant from the bulkhead on which the standard 16-ft. weir was located, is more nearly at the proper distance from the weir. Moreover, the up-stream piezometer was situated so far back from the standard weir as to be evidently disturbed somewhat by the entrance velocity of the water in the leading channel.

With the exception of Experiments Nos. 18, 19, 20 and 21, the readings at the experimental weir were taken from a piezometer placed horizontally across the channel of approach at a height of about 8 ins. above the bottom. In order to reduce the readings from this piezometer to the equivalent readings from the piezometer placed

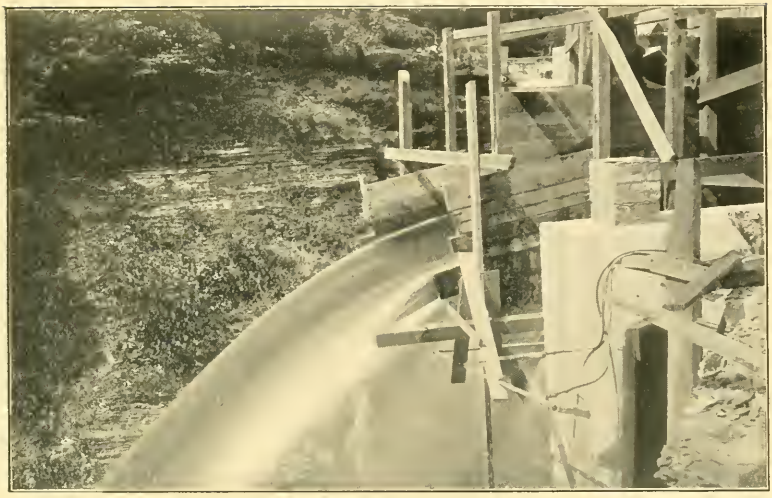


FIG. 1.—LOWER END OF CHANNEL. CORNELL UNIVERSITY HYDRAULIC LABORATORY.

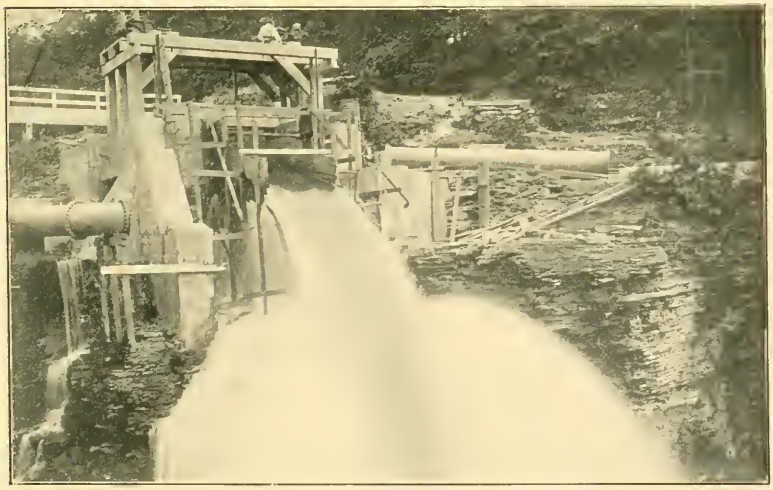


FIG. 2.—LOWER END OF CHANNEL. CORNELL UNIVERSITY HYDRAULIC LABORATORY.

flush with the bottom of the channel, a correction curve has been deduced in the following manner: In Experiments Nos. 18 and 19 observations were taken, both from the flush piezometer and from one situated 8 ins. above the bottom. Plotting the differences between the readings of these two piezometers corresponding to given heads on the standard weir as ordinates, and using the observed heads on the standard as abscissas, a mean curve has been drawn, from which a correction to be applied in any case can be read directly. This curve is shown in Fig. 6.

In regard to the use of this correction curve, it may be pointed out that the error resulting from the use of a piezometer placed otherwise than flush with the bottom or side of the channel is a function of the velocities in the channel of approach. Inasmuch as the discharging capacities of weirs of different sections vary greatly under the same heads, the velocity of approach at any given head will depend both

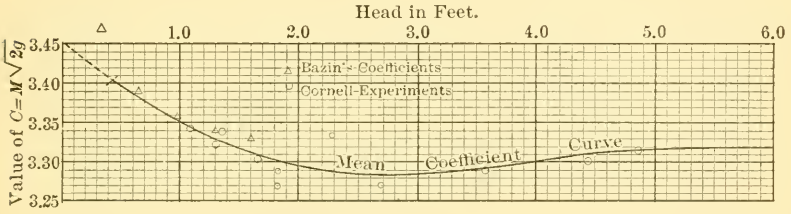


FIG. 7.

upon the height P of the weir and on the coefficient of discharge for the particular head and section of weir considered. Unfortunately, the data obtained were insufficient to enable the effect of these elements to be taken into consideration separately. Hence, the corrections obtained from the mean curve of Fig. 6 must be considered as approximate only. Some of the deviation of the experimental coefficients from the mean coefficient curves may undoubtedly be attributed to the uncertainty as to just the proper value of this correction.

Pages 267 to 284 show the coefficient curves finally fixed upon by the foregoing discussion for Cornell University Experiments Nos. 1 to 19, inclusive, and also the tabulations of the results. The coefficient curve for Experiments Nos. 20 and 21 on the standard sharp-crested weir is shown in Fig. 7. These curves are so self-explanatory as to render extended description unnecessary.

In the tabulations Column (1) shows the heads in feet, as read from the coefficient curves, Column (4) giving the discharge per lineal foot of crest in cubic feet per second. Columns (2) and (3) give the values of the coefficients M and C.

The photographs on Plate XIV show the lower end of the lower channel, as it appeared on the afternoon of June 3d, 1899, while experiments on the Rexford Flats section were in progress.

It is not the writer's intention to review extensively the results of the Cornell University Experiments at this time, any farther than to point out that they were, in the fullest sense, practical experiments.

In Experiments Nos. 7 and 8 an attempt was made to gain some idea of the effect of a rough surface on dams. In Experiment No. 7 the weir was of the usual form, with the crest constructed of planed matched pine, as already described, while in Experiment No. 8 the up-stream face of the crest was covered with $\frac{1}{4}$-in. mesh, galvanized wire screen. A comparison of these two experiments is very instructive. The upper limiting head of No. 7 was 4.996 ft. and of Experiment No. 8, 5.011 ft. For 5 ft. head, as determined from the curves, we have for Experiment No. 7, a discharge of 40.98 cu. ft. per second per lineal foot of crest, while for Experiment No. 8, 5 ft. head gives a discharge of 40.74 cu. ft. per second per lineal foot of crest. Similar comparisons at other heads show the effect of the wire screen to have been but slight. The result of this experiment was such as to lead to the conclusion that very little difference would be experienced in the flow over a dam after the first few months, during which time the planking, under the smoothing effect of the silt in flowing water, etc., may be expected to come substantially to the hydraulic condition of planed boards. Accordingly, as the time for completing the work was limited, no further determinations were made on this line.

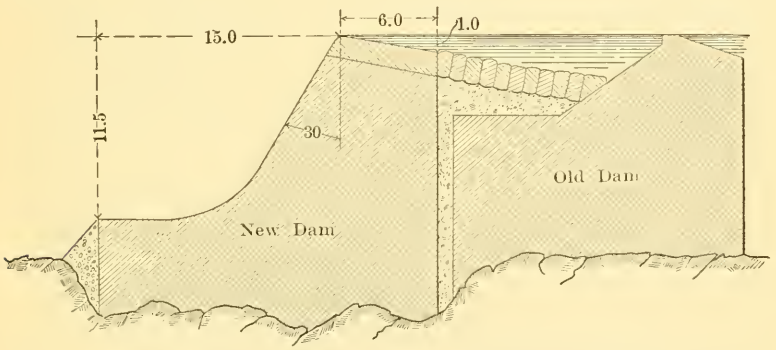
It is recognized, however, that but for the limitation of time under which the experiments were carried out, it would have been very desirable to have experimented somewhat farther on a number of additional forms of weirs, and it is to be hoped, in view of the vast practical importance of an accurate knowledge of flow over dams, that the Cornell University authorities will carry these experiments considerably farther, keeping especially well within the limits of actual practice in dam construction.

Application of Data to Cases in Practice.

We may now consider the application of the foregoing data to some of the dams at several of the gauging stations previously referred to.

Seneca River at Baldwinsville.—At Baldwinsville Station on Seneca River, there is a substantial masonry dam, as shown by Fig. 8. It was built in 1895, taking the place of an old crib dam located just above, as shown in the illustration. The crest is 423 ft. long, and is very nearly level. The catchment area of Seneca River, at Baldwinsville, is 3 103 sq. miles.

For most of the year flash boards, 1 ft. in height, are used on a portion of the crest. The flow over these has been computed by



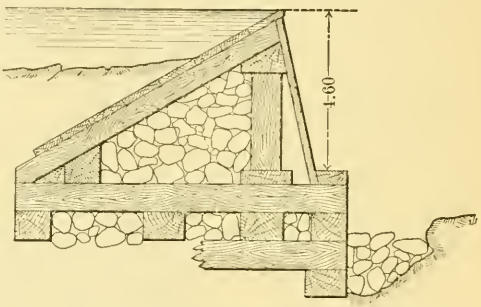
CROSS-SECTION OF DAM ON SENECA RIVER AT BALDWINSVILLE.

Fig. 8.

Francis' formula. The situation is somewhat complicated by the presence of the old dam. Taking into account the water cushion between the two crests, it was considered that Bazin's Series No. 115 would fairly apply, and, accordingly, the discharge curve was computed on this basis. The conditions here are so unusual that a special determination should be made as a check on the foregoing assumption. This was not done, during the Cornell University Experiments, for lack of time.

Oswego River at Fulton.—The catchment area above this dam is 4 916 sq. miles. There are extensive manufacturing establishments at the ends. The dam is a substantial masonry construction with a nearly vertical front, and with a back slope of 1 to 8. The

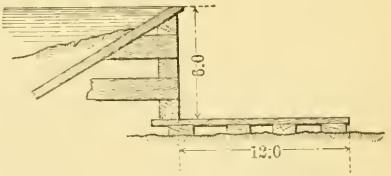
crest is slightly rounded. Bazin's section making the nearest approach to this is Series No. 117, which was recognized as being only an approximation. Cornell University Experiment No. 3 has furnished the data for working out a new discharge curve applying more nearly to the conditions of this dam than the original. It is believed that the revised curve gives the true discharge within a small percentage. At a depth of 2 ft. on the crest, the discharge, by the revised curve, is 9.8% less than by the original curve.



CROSS-SECTION OF DAM ON CHITTENANGO CREEK AT BRIDGEPORT.

FIG. 9.

Chittenango Creek at Bridgeport. — The cross-section of this dam is shown by Fig. 9. The catchment area above the point of gauging is 307 sq. miles, while the crest is 259.2 ft. in length. At the ends there are platforms over the bulkheads, and about 2.5 ft. above the main crest. The flow over these platforms when the water rises to their height, was originally computed by Bazin's Series No. 113, while Series No. 130 was applied to the main crest, shown by Fig. 9. The revised discharge curve for this dam is based upon Cornell University Experiments Nos. 1 and 10. The computed discharge, as per the revised curve, is—at a depth of 2 ft. on the crest—11.9% less than by the original curve.



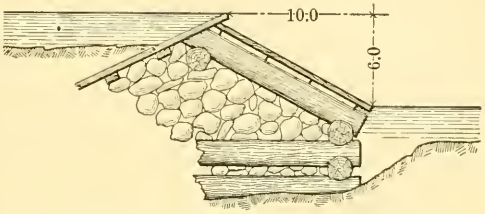
CROSS-SECTION OF DAM ON ONEIDA CREEK AT KENWOOD.

FIG. 10.

Oneida Creek at Kenwood.— The catchment area above this dam is 59 sq. miles. The crest is level, and is 79.4 ft. in length (see Fig. 10). The cross-section corresponds closely to Bazin's Series Nos. 130 or 135. The final discharge curve has been worked up from Cornell University Experiment No. 2.

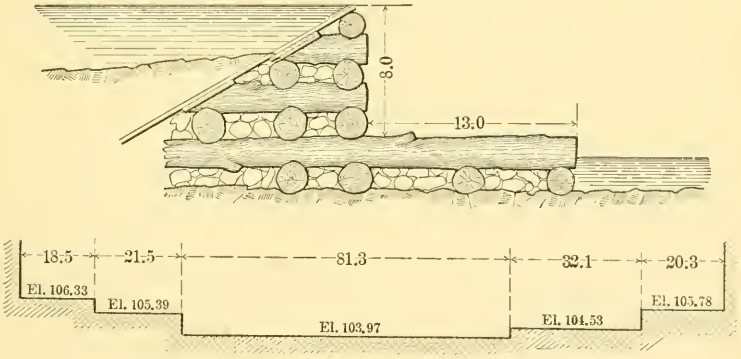
West Branch of Fish Creek at McConnellsville.—The catchment area above this dam is 187 sq. miles. The crest was originally quite irregular longitudinally, but was brought to a nearly uniform level by

spiking on strips of plank, which extended down the back or up-stream face. The length is 175.7 ft. (see Fig. 11). Bazin's Series No. 170 conforms in its general form closely to the cross-section, except that the projection of the planking of the back face over the front, forms an air space which has a disturbing effect on low flows. For minutely accurate results, on such a profile, special determinations should be made. Cornell University Experiment No. 7 has been used in preparing the final discharge curve.



CROSS-SECTION OF DAM ON WEST BRANCH OF FISH CREEK AT McCONNELLSVILLE.
FIG. 11.

East Branch of Fish Creek near Point Rock.—The catchment area above this dam is 104 sq. miles. The crest is 173.7 ft. in length, and is at several different heights, as shown in Fig. 12. Bazin's Series No. 130 applies closely. The final discharge curve is based upon Cornell University Experiment No. 1.

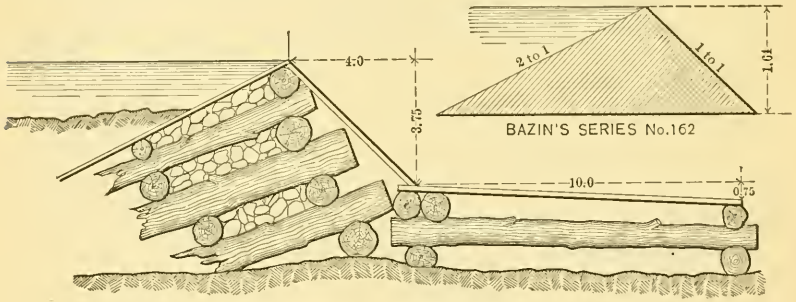


CROSS-SECTION AND PROFILE OF DAM ON EAST BRANCH OF FISH CREEK NEAR POINT ROCK
FIG. 12.

Mohawk River at Ridge Mills.—The catchment area is 153 sq. miles. The crest is 122.7 ft. long, and is at three different elevations. As shown by Fig. 13, the experimental and actual sectional profiles agree closely.

Bazin's Series No. 162 is of such a form that the discharge is nearly

uniform at all heads. The syphon action of the sloping front face begins at low heads and continues to act with indefinitely increasing heads. There is no point where marked changes in regimen occur, as with depressed and adhering nappes. The coefficient of Cornell University Experiment No. 6 agrees closely with Bazin's No. 162. A discharge curve, based upon Bazin's No. 162, varies at 2 ft. depth on crest, only 1.5% from the discharge curve by Cornell University Experiment No. 6. Crests of this general form are especially applicable wherever accurate records of flow are required. For reasons given by Bazin, in the preceding abstracted matter, crests of this general form, but with flat upper surface, should be avoided. On this point see page 263 preceding.



CROSS-SECTION OF DAM ON MOHAWK RIVER AT RIDGE MILLS, IN COMPARISON WITH BAZIN'S SERIES No. 162.

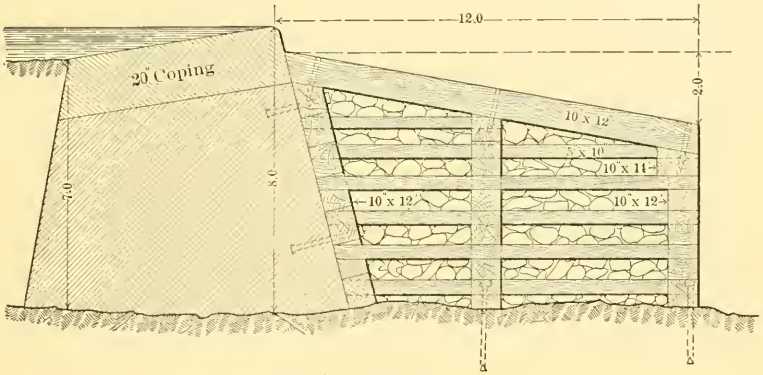
FIG. 13.

Mohawk River at Little Falls.—The catchment area at this place is 1 306 sq. miles. The dam is of well-built masonry, curved in plan, with a crest 181.7 ft. in length. There are two sections, with cross-sections corresponding to Cornell University Experiments Nos. 16 and 17, except that the dam has slopes on the front face for the two sections, 1 : 6 and 1 : 4, respectively. Inasmuch as there was full admission of air in the experiments, this fact would not affect the results, although the projection of the front face of the dam, itself, due to the slope may introduce disturbing elements in the flows, which would modify somewhat the results of Cornell University Experiments Nos. 16 and 17. The vertical front was used in the experiments in order to expedite the work.

The discharge curve worked out originally for this dam, was based upon Bazin's Series Nos. 117 and 135. A new curve, derived from

Cornell University Experiments Nos. 16 and 17, gives, for heads of 2 ft., 8.8% less flow than the original.

Mohawk River at Rexford Flats.—The catchment area at this place is 3 385 sq. miles. The dam is a substantial masonry construction with a timber apron, as shown by Fig. 14. The crest is 675 ft. long. The filling at the back of the dam makes, in effect, a long flat crest on the

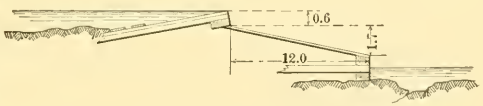


DAM ON MOHAWK RIVER AT REXFORD FLATS.

FIG. 14.

up-stream side. In the original discharge curve, Bazin's Series Nos. 117 and 141, were taken as applying best. This dam was experimented upon at Cornell University, the new discharge curve resulting therefrom agreeing very closely with the original curve. The variation in the two curves at 2 ft. depth on the crest is only about 1 per cent.

Oriskany Creek at Oriskany.—The catchment area here is 144 sq. miles. The crest is 214 ft. in length, and is at three different elevations (see Fig. 15). For the original discharge curve, a mean of the coefficients of Bazin's Series Nos. 117 and 141 were considered to apply best. A revised discharge curve, based upon Cornell University Experiment No. 14, has been worked out. At a depth of 2 ft. the new curve increases the discharge 1.7 per cent.



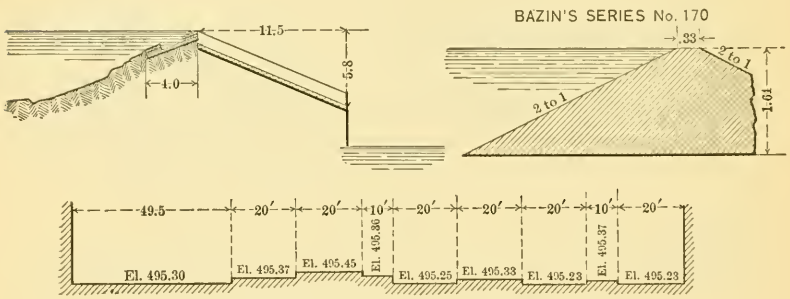
CROSS-SECTION OF DAM ON ORISKANY CREEK AT ORISKANY.

FIG. 15.

Oriskany Creek at Coleman.—This station is a little more than a mile above the dam at Oriskany, just described. The catchment area is

141 sq. miles, or 3 sq. miles (2.2%) less. Fig. 16 shows the cross-section in comparison with Bazin's Series No. 170, as well as the irregularities of the crest longitudinally. The remarks previously made as to flow, on Bazin's Series No. 162, apply to No. 170 and other sections of similar form. The disturbing effect of the departure from the theoretical form is unknown in this case, the same as for the dam on Mohawk River at Ridge Mills.

The object of establishing two stations on Oriskany Creek was to determine whether on dams of different forms, but with nearly the same catchment areas, the flows could be gauged closely enough to give fairly comparable figures. The following tabulation gives the



CROSS-SECTION AND LONGITUDINAL PROFILE OF DAM ON ORISKANY CREEK AT COLEMAN, IN COMPARISON WITH BAZIN'S SERIES No. 170.

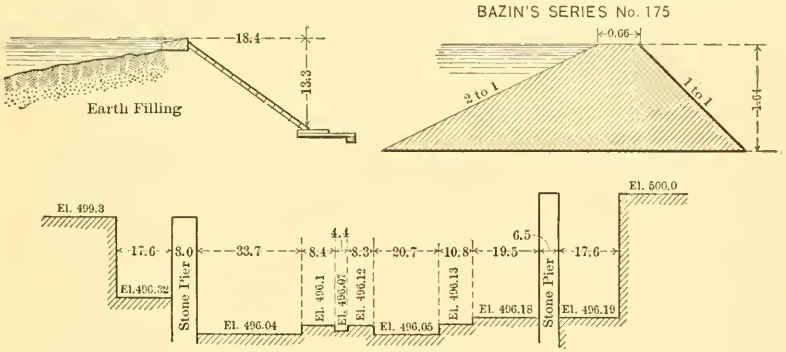
Fig. 16.

flows at Oriskany and Coleman, in cubic feet per second, for the months from November, 1898, to February, 1899, inclusive. During the frozen months of this period, the ice was kept clear for several feet back from the crest of each dam. The results show fair agreement, and indicate, that, even when one of the cases is complicated, as at Coleman, by discharge through several water wheels, comparable results may still be gained.

Month.	Discharge at Oriskany.	Discharge at Coleman.
November	327	306
December	327	335
January	295	297
February	291	283

Saquoit Creek at New York Mills.—The catchment area here is 52 sq. miles. The crest is as shown by Fig. 17. Bazin's Series No. 175 is taken as applying best to the main section. For the flash boards at the end sections, Francis' formula has been used.

West Canada Creek at Middleville.—The catchment area above this dam is 519 sq. miles. The crest is 330.5 ft. in length, and is leveled up, as described for the dam at McConnellsville (see Fig. 18). The original discharge curve was based upon Bazin's Series No. 170. A new curve based on Cornell University Experiment No. 15 (Rexford Flats section with rounded corner) gives 23% less discharge at 2 ft. depth than the original. The writer considers that this section should be specially determined, for accurate results, and the foregoing



CROSS-SECTION AND LONGITUDINAL PROFILE OF DAM ON SAQUOIT CREEK AT NEW YORK MILLS, IN COMPARISON WITH BAZIN'S SERIES No. 175.

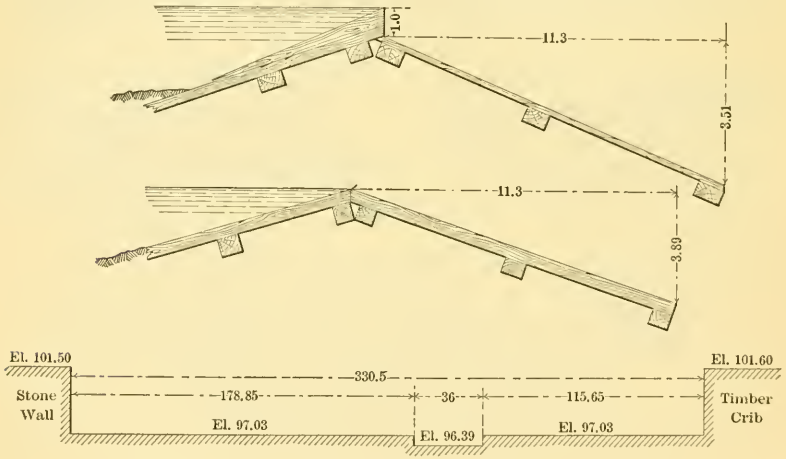
FIG. 17.

discrepancy is cited merely to show how useless it is, if accurate results are required, to apply the nearest form at hand. The whole study shows that, frequently, what appears at first sight to be relatively unimportant produces very marked changes in discharge.

Cayadutta Creek near Johnstown.—The tributary catchment area is 40 sq. miles. The main section, Fig. 19, is quite different from any of the dams thus far considered. Bazin's Series No. 130, was taken as being nearer than any other, while Series No. 115 was applied to the bulkheads at the ends. Where a high degree of accuracy is required for gaugings over a nondescript section of this sort, special experiments must be made.

Schoharie Creek at Fort Hunter.—The catchment area above this dam is 947 sq. miles. As shown by Fig. 20, it has a sectional profile similar to those of the dams on Oriskany and West Canada Creeks. The original discharge curve was based upon Bazin's Series Nos. 117 and 141. A re-computation, using Cornell University Experiment No. 14, gives substantially the same curve.

The foregoing account of several applications of the new views as to flow over dams has been made as concise as possible in order not to lengthen this paper unnecessarily. Matters of interest relating to the leakage of dams and flumes, methods of computing discharge through nondescript water wheels, and many other questions, are pur-



CROSS SECTIONS AND PROFILE OF DAM ON WEST CANADA CREEK AT MIDDLEVILLE.

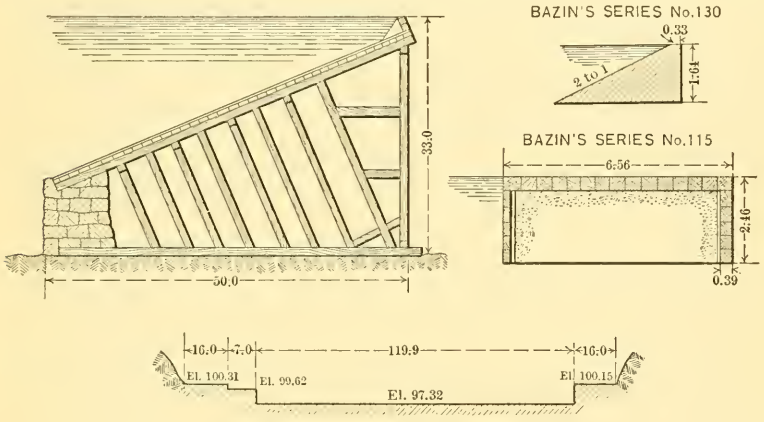
FIG. 18.

posely left untouched, in order to discuss more thoroughly the main question of how to compute the flow over dams. The Schoharie Creek dam may be especially mentioned as one with considerable leakage, and which is used here for illustrative purposes only.

In passing, it may be remarked that one result of the Cornell University Experiments was to show that the discharges, as per Bazin's Series Nos. 130 and 135, were much too high, especially at the considerable heads occurring at several of these gauging stations. The reasons for this are found, apparently, in the high discharges accompanying the depressed and adherent nappes, which occur at the low

heads Bazin experimented with. This, Bazin himself has pointed out, in the matter abstracted from his last paper, on a previous page. The great difference, however, is only fully realized when we carry out comparable experiments to the heads used at Cornell University.

The different methods of experimentation may also be taken into account. Bazin usually began with a low head, gradually increasing to the higher; whereas, at Cornell University, in all the experiments, the high heads were run first, and were gradually reduced. In both cases, the established regimen of flow, whatever it may have been, was continued longer than would have occurred under the contrary condition, the coefficients for the two states lapping by one another.* The



DAM ON CAYADUTTA CREEK NEAR JOHNSTOWN, ETC.

FIG. 19.

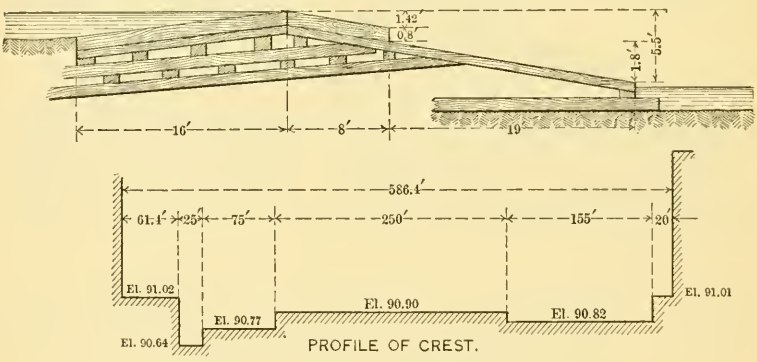
conclusion under this head is, therefore, that for a rising stream the discharge at or near the critical point of change, may be appreciably different from the discharge for a falling stream at about the same point.

By way of illustrating the method of computation used, we may discuss the computation for Schoharie Creek dam. Table No. 4 shows how the data for the discharge curve for this dam have been arrived at, the coefficients used therein being derived from Cornell University Experiment No. 14. To begin with, zero of the crest gauge is at elevation 90.68 ft. The crest itself divides into a series of sec-

* As to just the condition of the nappes for Bazin's Series Nos. 130 and 135, and several similar sections, see the tabulations of Bazin's Series, on preceding pages.

tions, with elevations as shown on the longitudinal profile of Fig. 20, and which are designated in the table, by the letters *A*, *B*, *C*, *D*, *E*, and *F*.

The method of procedure for the computation of points for the discharge curve is as follows: The average elevation of each section of dam—*A*, *B*, *C*, etc.—having been computed with reference to zero of the crest gauge, the depth of water flowing over each section, corresponding to a series of readings on the gauge, was deduced and tabulated, as shown. Thus, for section *A*, we have, in Column (4), head on section in feet, and so on for the other sections. Column (4) also includes the discharge per lineal foot of crest, for heads ranging from 0.2 ft. up to 8.0 ft., together with the total flow per section, for the same heads. These computations are made on the basis of



SECTION AND PROFILE OF SCHOHARIE CREEK DAM,

FIG. 20.

no end-contraction at the ends of the sections. The summation at the foot of the table represents the total flow over the dam, and ranges from 26 cu. ft. per second, for a head of 0.2 ft. to 41 182 cu. ft. per second, for a head of 8.0 ft. The discharge curve is constructed by plotting the final footings, with the heads on the crest in feet as ordinates, and discharges in cubic feet per second, as abscissas.

The foregoing general method has been applied, with necessary variations to fit each special case, to all the gauging stations herein referred to.

On examining the values of $C = m\sqrt{2g}$ in the coefficient tables given herewith, the great range in discharge, not only for different

TABLE No. 4.—Showing Method of Computing Discharge Curve for Dam on Schoharie Creek at Fort Hunter.

Discharge Coefficients are from Cornell University Experiment No. 14.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
Section.	Length, in feet.	Elevation, in feet.	Head above gauge zero.	0.2	0.6	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0
			Head on Section *A*, feet.....	.....	0.27	0.67	1.67	2.67	3.67	4.67	5.67	6.67	7.67
A.....	20	91.01	Discharge per foot of crest.	.....	0.37	1.62	6.82	14.13	22.73	32.62	43.62	55.80	68.30
			Total flow over section.....	.....	7.40	32.40	136.00	283.00	455.00	652.00	872.00	1 116.00	1 366.00
			Head on Section *B*, feet.....	0.06	0.46	0.86	1.86	2.86	3.86	4.86	5.86	6.86	7.86
B.....	155	90.82	Discharge per foot of crest.	0.06	0.90	2.40	8.20	15.70	24.65	34.65	45.90	58.00	70.85
			Total flow over section.....	9.30	139.50	372.00	1 271.00	2 434.00	3 821.00	5 371 00	7 115.00	8 990.00	10 982.00
			Head on Section *C*, feet.....	.....	0.38	0.78	1.78	2.78	3.78	4.78	5.78	6.78	7.78
C.....	250	90.90	Discharge per foot of crest.	.....	0.70	2.15	7.60	15.10	23.90	33.95	45.00	57.05	69.80
			Total flow over section.....	.....	175.00	538.00	1 900.00	3 775.00	5 975.00	8 488.00	11 250.00	14 263.00	17 450.00
			Head on Section *D*, feet.....	0.11	0.51	0.91	1.91	2.91	3.91	4.91	5.91	6.91	7.91
D.....	75	90.77	Discharge per foot of crest.	0.10	1.10	2.62	8.46	16.20	25.05	35.20	46.50	58.50	71.50
			Total flow over section.....	7.50	83.00	196.00	634.00	1 215.00	1 879.00	2 640.00	3 488.00	4 388.00	5 363.00
			Head on Section *E*, feet.....	0.24	0.64	1.04	2.04	3.04	4.04	5.04	6.04	7.04	8.04
E.....	25	90.64	Discharge per foot of crest.	0.36	1.77	3.28	9.40	17.20	26.40	36.60	48.05	60.30	73.30
			Total flow over section.....	9.00	44.00	82.00	235.00	430.00	660.00	915.00	1 201.00	1 508.00	1 833.00
			Head on Section *F*, feet.....	.....	0.26	0.66	1.66	2.66	3.66	4.66	5.66	6.66	7.66
F.....	61.4	91.02	Discharge per foot of crest.	.....	0.36	1.60	6.80	14.10	22.70	32.58	43.58	55.75	68.20
			Total flow over section.....	.....	22.00	98.00	417.00	865.00	1 394.00	2 000.00	2 676.00	3 423.00	4 187.00
Total flow over dam, in cubic feet per second.....				26.00	471.09	1 318.00	4 593.00	9 002.00	14 184.00	20 066.00	26 602.00	33 688.00	41 182.00

Elevation of zero of crest gauge = 90.68 ft.

forms of dams, but for varying heads, becomes apparent. These tables indicate that the making of accurate gaugings over dams demands considerable skill in the application of the available information. For minutely accurate results, special experimentation is, in many cases, indispensable.

As regards experiments on dams, the Cornell University Hydraulic Laboratory can hardly be improved upon, and the University authorities deserve the sincere thanks of every hydraulician, for furnishing an equipment of this character. It is to be hoped that data of flow over dams may be greatly extended there in the next few years.

In concluding the paper, the writer may remark that the studies herein discussed were, in reality, only a side issue of the entire investigation of the water-supply problems carried out for the United States Board of Engineers on Deep Waterways. For whatever deficiencies may appear, the writer hopes he may be pardoned, because of the time limit set by the Board, which was, the completion of everything within one year. This condition compelled a strictly business-like administration and the omission of much purely scientific detail which, with more time available, it would have been very pleasant to pursue somewhat farther. It was necessary, indeed, under the requirements of the Board, to be first of all a business man—driving the matter in hand along rapidly to a final conclusion—and only indulging in pure scientific work so far as this did not conflict with definite progress from day to day.

As regards the Cornell University authorities, the conditions are different, and they will without doubt ultimately supply the engineering profession with far more extended knowledge of the flow of water over dams, especially at high heads, than is now possessed.

The total cost of this set of experiments, including materials, common labor, carpenters, engineering assistants, draughtsmen, stenographer and time of writer did not exceed $1 800. This figure does not include either Professor Williams' time or cost of gauges, which were paid for by the University as part of the permanent equipment, but it includes all payments on account of these experiments by the United States Board of Engineers on Deep Waterways.

The writer's thanks are due to Professor E. A. Fuertes, Director of Cornell University College of Civil Engineering, for many courtesies received during the progress of the study.

DISCUSSION.

GEORGE Y. WISNER, M. Am. Soc. C. E. (by letter).—In the investigation for developing a project for a deep waterway from the Great Lakes to the Atlantic, which was commenced by the United States Board of Engineers on Deep Waterways in the fall of 1897, it became evident, from the start, that the existing data relative to the flow of water over dams were inadequate for the accurate determination of river discharge where the depth on the crest of the dam was much over 1.5 ft. Mr. Wisner.

The uncertainty as to the value of the coefficients of weir formulas which should be used for dams of different cross-sections, and for different depths on the crest, made it apparent that additional investigations would be necessary before satisfactory estimates could be made of the value of water-power rights which may be modified, or of the amount of slope walls and bank protection which would be needed between the limits of the high and low-water stages of the proposed waterway.

At first, it was thought that extended observations with modern current meters at several of the principal dams in question would be necessary, but since the coefficient of the weir formula varies greatly with the shape of the dam, and as there are but few of the dams on the Oswego, Mohawk and Hudson Rivers which have similar cross-sections, such a method would have been incomplete and unsatisfactory, and was not attempted.

In the fall of 1898 the experiments of H. Bazin, published in the *Annales des Ponts et Chaussées*, became available, and established the coefficients for a great variety of different-shaped weirs, but, unfortunately, for depths of less than 1.5 ft. on the crests.

Soon after the appearance of Bazin's final paper, the writer entered into correspondence with Professor Williams, and later with Mr. Rafter, with a view to utilizing the Hydraulic Laboratory at Cornell University for the extension of the Bazin experiments to greater heads. The work was finally ordered in the following April, Messrs. Rafter and Williams acting in conjunction. Mr. Williams' special work was the installation of the necessary measuring apparatus and the measurement of the heads on the weirs during the experiments.

It was not expected that a high degree of precision would be attained, but it was believed the results would suffice for determining with all needful accuracy the floods of the Mohawk and Hudson, from the observed heights on the several dams, and also for the preliminary design of the Lake Erie Regulating Works. The experiments have answered these requirements, and are exact enough for application to any similar practical case.

In authorizing these experiments, the Board of Engineers on Deep

Mr. Wisner. Waterways insisted that the standard sharp-crested weir should be thoroughly calibrated for all depths on the crest for which used. This, however, the observers were unable to accomplish in a satisfactory manner, but it is to be hoped this will be done in time to embody the results in this discussion.

Referring to the mean coefficient curve of the standard weir shown by Fig. 7, it will be noted that for heads of from 2 to 3 ft., the coefficient is a minimum. So far as the writer is aware, there is no good reason for this, and it is probable that the peculiar shape of the curve may be due to incorrect calibration of the standard weir, and to the effect of velocity of approach and to side walls of a narrow flume for which the correction made is, apparently, largely a matter of judgment.

An examination of the curves on pages 267 to 284, inclusive, shows the same peculiarity for a large percentage of the experiments, and a study of the data and results indicates that incorrect determination of the effect of velocity of approach at the experimental weir and unknown resistances of the side walls of a narrow flume are the principal causes. Comparing the curve on page 284 with that in Fig. 7, there is apparently no reason why one should be a regular curve and the other not, and, as observations made subsequent to those discussed in the paper indicate that the former is correct, it is a fair presumption that additional observations will very likely modify the shape of the latter.

Mr. Williams. GARDNER S. WILLIAMS, M. Am. Soc. C. E.—It would afford the speaker great pleasure were he able to give nothing but commendation for the work of the author. Unfortunately, he is compelled to differ from the opinions expressed in the paper, on some rather essential points. He is fully aware that any criticisms that may be made, so far as they relate to the execution of the experiments, will be criticisms upon himself, and, in extenuation, has only to say that the institution which he has the honor to represent is searching for the truth, and will be glad, as occasion comes, to point out its own errors as well as those of others. It will welcome any criticisms, any suggestions, any instructions.

At the outset of the investigation described, it was decided that as the work was intended to be an extension of that of Bazin, the conditions of his experiments would be conformed to as nearly as possible, and his formula be used in the reductions. For this reason the experimental weirs were made 2 m. long, and the piezometers for reading the heads were located at distances from the experimental weirs bearing approximately the same ratio to their height that the distance to the location of Bazin's point of reading head bore to the height of his weirs. Bazin's formula* differs from the formulas of Francis, Fteley and Stearns, and Hamilton Smith, in that it provides

* { *Annales des Ponts et Chaussées*, October, 1888.
{ *Proceedings*, Engineers' Club, Philadelphia, 1890, Vol. vii, p. 251 *et seq.*

for the effect of velocity of approach in its coefficient; whereas all the others require that the observed head shall be corrected for the velocity of approach, so that, while the Francis coefficient, 3.33, is to be applied to the head after it is thus corrected, the Bazin coefficient is to be applied to the head as it is observed. Mr. Williams.

The statement of the author that by reference to Table No. 2 it will be seen that there is very little change in the coefficient after a height of weir of 6½ ft. is reached, also needs some decided modification, because the coefficient changes as the velocity of approach changes, and while the statement is true with the comparatively low heads, up to 1.6 ft., included in the table, it is not true with higher heads, as, for example, with a head of 100 cm., that is, about 3⅓ ft., on a weir 11 ft. high, the discharge is about 0.6 of 1% greater than it is on a weir 13 ft. high, according to Bazin's formula.

Of course, it is fully realized that in speaking of a matter of 0.6 of 1% in a weir measurement it is getting down rather fine; but we are to consider that accuracy in the measurement of water for power purposes or for consumption and accuracy in the design of a crest to discharge the flood volumes of a stream, are two different things. In the latter case, if one comes within 5%, he is doing very well. In the former, one should get down as near to 1% as possible.

The author, apparently, makes a misstatement on page 292,* where he says that n—which was originally designated by the Greek letter μ by Bazin—is a coefficient which depends upon the height of the weir; now that is less than half the truth. It depends upon the height of the weir and the head over the weir,† and it is given quite accurately by the formula $\mu = 0.405 + \frac{0.00984}{h}$ for English units. That is, it is given satisfactorily by a formula which does not involve the height of the weir at all. In other words, in speaking generally, the coefficient μ varies with the head over the weir and not with its height. The coefficient m varies with the head over the weir and the height of the weir, and m is the factor which enters finally into Bazin's formula, the formula being virtually made up of three formulas, first a formula for μ, then one for m, which involves μ, and then one for Q which involves m.

In the reduction of Experiments Nos. 20 and 21 on pages 293 and 294, if the speaker understands the author correctly, a curve has been plotted for the discharge of the upper weir by Bazin's formula, as given on page 292, which contains the coefficient providing for velocity of approach. From this the author has taken a value for the discharge. He has then from that discharge computed the velocity of approach, added to the observed head the correction for the head due to this velocity of approach, taken the value of Q for this increased head

*Since this discussion was received, the statement in the original paper has been changed and now reads:
n = a coefficient which depends on p and h (p = height of crest of weir above bottom of channel of approach; and h = observed head on crest).

† *Annales des Ponts et Chaussées*, Oct., 1888, p. 445, and *Proceedings*, Engineers' Club of Philadelphia, 1890, p. 308.

Mr. Williams. from the Bazin curve again, and he says that generally two applications of the process were sufficient. In the speaker's judgment, inasmuch as each application was adding a velocity head that did not belong there, it seems that two applications should have been sufficient. If the author desired to determine the discharge for a weir under this head with no velocity of approach, the correction should have been subtracted, not added, and if he wished to obtain the head required on a weir with no velocity of approach to deliver the same quantity of water the correction for velocity of approach should have been made but once.

It is possible that the speaker has misunderstood the author's method of reduction, but he has taken pains to refer these statements to several others, conversant with Bazin's formula and with hydraulics in general, and it has been generally agreed that the language is misleading, if the process has not actually been so. That is, the language simply means that instead of using the observed head h which should have been used with Bazin's formula, the head $h + h_a$, as in the Francis formula, was used, the result being that the computed discharge of the standard weir by Bazin's formula is thereby made too large. It may be pertinent to enquire why, if the author deemed a correction for velocity of approach necessary at the upper weir, he did not also apply one at the lower weir where the velocity was several times as great. The author does not appear to have understood clearly the import of the discussion by Messrs. Fteley and Stearns upon the position for reading heads upon a weir, judging from the statement on page 294 that: "Messrs. Fteley and Stearns have pointed out that for standard sharp-crested weirs the head should be measured about 6 ft. back from the crest," for, by turning to the paper by Messrs. Fteley and Stearns,* the following statement will be found :

"The head, if measured outside of the angle of pressure, should be taken far enough up stream from the weir to represent the height of the water surface above the beginning of the surface curvature, *i. e.*, at a distance from the weir equal to $2\frac{1}{2}$ times its height above the bottom of the channel."

Our weir was 13 ft. high, and the head, according to this, should have been read about 32 ft. up stream.

This brings us quite properly to the subject of weir experiments in general, and in the discussion of any hydraulic problem it is well to go back to the beginning and find out how much we really know about the thing in hand. We are dealing with weirs with end contractions suppressed, and so far as experiments have gone upon such weirs of a sufficient size to be compared with those which are discussed in this paper, in which the discharge has been measured volumetrically, the entire series of experiments is embraced in three investigations shown in Table No. 5.

* *Transactions*, Am. Soc. C. E., Vol. xii, p. 47.

Mr. Williams.

TABLE No. 5.—Experiments upon Weirs with End Contractions Suppressed, in which the Discharge was Measured Volumetrically.

Observer.	Number of experiments.	Length of weir, in feet.	Height of weir, in feet.	Range of head, in feet.
James B. Francis....	17	9.995	4.60	0.73620 to 1.0600
Fteley and Stearns...	30	4.999	3.56	0.0746 " 0.8193
" " ...	10	18.996	6.55	0.4685 " 1.6038
Henry Bazin.....	67	6.562	3.72	0.194 " 1.012
" "	38	3.281	3.72	0.188 " 1.338
" "	48	1.640	3.296	0.191 " 1.779

It will be seen that Bazin's first series included nearly as many experiments as those of all the other investigators, and that, altogether, he has given us three times as many determinations of the flow over suppressed weirs volumetrically as the others have.

There is an important distinction between the methods of measuring head in Bazin's experiments and in the experiments of the American investigators. The latter adopted a position for reading the head 6 ft. up stream from the crest of the weir and about its level. Bazin read it 16.3 ft. up stream and at the bottom of the channel. The American experimenters took the water through a small opening in the side, in no case more than $\frac{3}{4}$ sq. in. in area, which communicated with a pail in which the surface was read by a hook gauge. Bazin used an opening 4 ins. in diameter which communicated to a chamber built alongside of his canal in which the head was read by a hook gauge. Now, it will be realized at once that it is to be expected that the velocity of the water flowing toward the weir would be greater at the American position than it would be 10 ft. further up stream, and, as any increase of velocity head or of velocity means a corresponding decrease of pressure head, it may be expected that for the same observation, if the head were measured at the American position, it would appear to be lower for the same discharge than if it were measured at Bazin's position. Therefore, for a given head, we should expect that Bazin would show a less discharge than would the American investigators.

Some may be inclined to doubt the importance of the variation in position in reading heads. Upon that point it may be said that in some investigations carried on last summer, the head was read directly at the crest of the weir by means of a tube set in the weir itself and communicating with the crest by small openings 6 ins. apart. These openings were about $\frac{1}{4}$ in. in diameter, and were bored vertically at the exact crest of the weir, which was the section adopted by the United States Board of Engineers on Deep Waterways for the proposed regulating weir on Lake Erie, *i. e.*, No. 19 of the author's series. At the

Mr. Williams

time that these heads were read a tape was tacked upon the wall of the canal vertically at the crest, so that the top of the sheet could be read thereon at the same time that the pressure in the piezometer along the crest of the weir was read. With a head up stream of 97 cm. the tape read 70 cm. and the piezometer at the crest read 38 cm. There is the effect of velocity upon the head. The piezometer set on the crest of the weir read hardly more than half the depth of water which was actually flowing over the weir at that time, and as there were, altogether, somewhere in the neighborhood of thirty or forty experiments involving the crest piezometer, it may be affirmed that this was not an erratic observation.

TABLE No. 6.—Comparison of Observations and Formulas of Suppressed Weirs.

No.	Observer.	Weir.		Observation.		Discharge by Formula.			
		Length. Ft.	Height. Ft.	Head. Ft.	Discharge. Cubic feet per second.	Francis. Cubic feet per second.	Fteley & Stearns. Cubic feet per second.	Smith. Cubic feet per second.	Bazin. Cubic feet per second.
1...	J. B. Francis.....	9.995	4.60	0.9760	32.436	32.290	32.300	32.406	32.784
2...	Fteley & Stearns.....	18.996	6.55	1.4546	112.066	111.890	112.054	112.240	112.550
3...	" "	18.996	6.55	0.4685	20.178	20.306	20.175	.....	20.890
4...	" "	4.999	3.56	0.8118	12.466	12.307	12.459	12.497	12.557
5...	" "	4.999	3.56	0.4569	5.199	5.162	5.212	.....	5.326
6...	H. Bazin.....	6.562	3.72	0.9794	21.930	21.418	21.478	21.715	21.928
7...	"	6.562	3.72	0.5644	9.533	9.306	.....	.....	9.533
8...	"	1.640	3.296	1.0158	5.754	5.664	5.689	5.961	5.753
9...	"	1.640	3.296	0.5332	2.2005	2.1346	.....	.....	2.2002

2 Excess by Bazin's formula over Fteley & Stearns' measurement for $h = 1.4546$, is 0.43%
1 " " " " " Francis " " $h = 0.9760$, " 1.07%
4 " " " " " Fteley & Stearns " " $h = 0.8118$, " 0.71%
3 " " " " " " " " " $h = 0.4685$, " 8.52%
5 " " " " " " " " " $h = 0.4569$, " 2.44%
8 " " " measurement over Francis' formula " $h = 1.0158$, " 1.59%
6 " " " " " " " " $h = 0.9794$, " 2.39%
7 " " " " " " " " $h = 0.5644$, " 2.44%
9 " " " " " " " " $h = 0.5332$, " 3.09%

It may be said further, that, since this condition exists, it is possible to use such a form of weir as a Venturi meter, particularly when submerged, and there is no doubt that a series of coefficients for a weir of the form of the United States Deep Waterways Section might be given as such a meter that would compare quite favorably, in accuracy, at least with the coefficients given by the author for the various irregular weirs. Time does not now suffice to go into this to its fullest extent. It may be said, however, that as the crest is submerged the difference between the reading of the piezometer and the tape decreases, but they do not become equal up to 3-ft. heads, nor does the reading of the piezometer at the crest become equal to the depth of submergence within

submergences of 4 ft. Now, as stated before, Bazin's formula should be expected to give the higher head for a given discharge or a lower discharge at a given head than those of the American investigators; but when Bazin's formula is applied to the experiments of the American investigators, in which the discharge was measured, it universally gives a higher discharge than was observed. When the American formulas are applied to the American experiments they fit excellently; but when they are applied to Bazin's experiments they give a lower discharge than was observed. When we apply Bazin's formula to his own experiments, it fits most excellently, as is shown in Table No. 6. Mr. Williams.

Now, what is the meaning of this? Either Bazin had a different kind of water from that which the American experimenters had, or one or the other has done the better work. The question comes home at once: Which? Bearing upon this point, a criterion has been applied, the best that has occurred to the speaker thus far, to determine whether the experiments of the several investigators were homogeneous in themselves; that is, whether they would coincide or whether they would show erratic variations from one side to the other of some mean. The criterion was to take the measured Q's and from them to derive an $n^{\frac{2}{3}} h$, which, when plotted as an abscissa with the observed head as an ordinate, would give a straight line if n were constant. Of course, since n is not constant, but increases with the head at the higher heads, it does not give a straight line, but the variation is not great for the range of the experiments. That criterion showed clearly that the results of the experiments were homogeneous in themselves, and indicated a high degree of relative accuracy. That is, if one was right the other was right in the same series. Pains were then taken to study particularly the arrangement by which the quantity was measured, and the conclusion has been that the devices used by Francis and by Fteley and Stearns for starting and stopping the flow and also for determining the height of water in the measuring basin were more delicate, and capable of more accurate work than were those of Bazin, so that, patriotism aside, it seems that greater confidence may be reposed in the observations of Francis and of Fteley and Stearns than in those of Bazin, although the latter has made three times as many as the others. As already stated, the Francis formula gives results below Bazin for the lower heads, but the discharge curves of the two formulas cross at a head of about 1.4 ft. on a weir 11 ft. high, and above that Bazin gives lower discharges. Comparing the Francis formula with the discharges observed by Fteley and Stearns at the higher heads there is some evidence that the Francis formula gives too low results with such conditions, and therefore it seems that Bazin's formula is probably on this account the less accurate at the high heads.

Whether or not all wish to agree with the deductions as to the effect of velocity of approach or velocity past the openings and as to

Mr. Williams. the relative reliability of the work of the various investigators, they will probably agree that if Bazin's formula is to be used, the head should be measured as Bazin measured it, and if the Francis formula is to be used, the head should be measured as Francis measured it. But the question naturally arises, what difference does it make? A few days ago the speaker had the privilege of performing some experiments to see what difference it made whether the head was measured one way or another. The weir in question was a small decimal over 20 ft. in length. Its height was 5.85 ft. above the channel of approach. End contractions were suppressed. At a point 10.3 ft. up stream from the weir there was a pipe, 1 in. in internal diameter, set 1 ft. above the bottom of the channel of approach, transversely to the direction of flow. This pipe was perforated on its bottom with holes about $\frac{3}{16}$ in. in diameter every 3 ins. in its length. One end of this pipe was connected by means of a $\frac{3}{4}$-in. pipe and a $\frac{3}{4}$-in. hose, to a hook-gauge pail, which was set in a recess in the wall at a point about 6 ft. up stream from the weir. At the other side of the channel of approach was a similar recess. This transverse pipe was a device which was ordinarily used for measuring the head upon this weir, which, it may be said, incidentally, is a somewhat important one. As a result of the investigations at Cornell, the reliability of a measurement taken in that way was questioned; and, in order that there might be no mistake in the important work which was in hand, the plate which formed the side of the recess on the opposite side of the canal was tapped through at a point 6 ft. from the crest of the weir and 0.35 ft. below it, and a $\frac{3}{4}$-in. iron pipe screwed in, the face of which was filed off flush with the side of the channel. A wooden plug was then driven into the pipe and smoothed off flush; in this a $\frac{1}{4}$-in. hole, perpendicular to the side of the channel, was bored, thereby nearly reproducing the device used by Francis and by Fteley and Stearns. The transverse pipe was connected through the side of the canal to a second hook-gauge pail in the same chamber. The hook-gauge pail was removed, and in its stead was connected one of two glass tubes of $\frac{3}{4}$-in. inside diameter, which were mounted rigidly on a board in front of a common scale, divided in 2 mm. divisions. The other tube was connected to the new, or what will be designated as the Francis, piezometer. Two portable hook gauges were then clamped to the crest of the weir, and the water was raised to within about $\frac{1}{4}$ in. of the crest, and by measuring from the crest with the portable hooks the reading of the scale and of the permanent hook gauge for the crest of the weir, were determined by water level, so that there might be no mistake as to the setting of the instruments.

A series of investigations was then made to determine whether the tube which was connected to the transverse piezometer read in correspondence with the hook gauge, and it was found to do so with remarkable constancy for a wide range of head, the difference being

that the tube showed continuously about 0.003 ft. more head than the hook. It was therefore considered that any difference which appeared in the readings of the two glass columns would show the difference in head by the two piezometers. Omitting further detail, it was found that at low heads the transverse piezometer read high, and that at high heads it read low, and with a head of 2 ft. over the weir it made 2½% difference in the discharge whether the reading was taken by the Francis piezometer or by transverse piezometer, the discharge by the Francis piezometer being 2½% greater than by the transverse piezometer. That will give an idea of the importance, when using any formula, of measuring the head as it was measured when the formula was devised. If the head is measured some other way the formulas may or may not apply. Unfortunately, at Cornell, no provision was made in the construction of the plant for measuring the head upon the standard weir by either of the recognized methods. It was, therefore, necessary to resort to some other means, and, without knowing positively what the result would be, the transverse piezometer, which has been described by the author, and which coincides quite closely with that at the weir, which has just been discussed, was adopted. There was some suspicion when it was put in that it might lead to trouble, and it was expected that such checks could be made on the work as it went along as to detect such an error at once if it should occur. But the great pressure which was brought to bear to hurry the experiments, and the other duties which were demanded of the investigators, prevented the working up of those experiments, even the first of them, until the series was nearly completed. Then it became apparent that there was something wrong with the piezometers, and, accordingly, there was set, alongside the one at the lower weir, another, which was flush with the bottom of the channel of approach, and which probably coincided quite closely with Bazin's opening in the side of the canal at the bottom, although, of course, it did not coincide exactly. It was then found that, at the highest heads used, the difference in head, as measured by the two piezometers, amounted to about 10 cm., or about 0.3 ft., which means considerable in the discharge of the weir. The author has given, in Fig. 6, a correction curve which he applies in these experiments, and which is probably the best that could be done under the circumstances. The discovery of such an error at the lower weir led at once to the conclusion that there must be something the matter with the piezometer at the upper weir also, but time did not permit an investigation previous to the completion of the series of experiments described. As the speaker was, at the close of this investigation, requested by the Board of Engineers on Deep Waterways to continue the work by an investigation of the discharge over the "Deep Waterways" Section, so-called, Section 19, a rounded crest with a 45° up-stream slope, at various degrees of submergence, and also

Mr. Williams.

Mr. Williams. with free discharge, it gave an opportunity to make some further investigations upon the foregoing question, and in order to measure the head in another way at the standard weir three pipes were set longitudinally, that is, parallel to the direction of the flow of the water at a point 6 ft. above the bottom of the canal and 28 ft. up stream from the weir and with their down-stream ends projecting about 6 ins. down stream from the plane of the old up-stream transverse piezometer. These pipes were 6 ft. long, $\frac{3}{4}$ in. in diameter, nominally, and were perforated for 1 ft. at their down-stream end with holes on the quarter, so that there were around the pipe rings of four holes, each 1 in. apart for a distance of 1 ft. at the lower end. These pipes were connected together at the down-stream end, the up-stream end being plugged. A $\frac{3}{4}$-in. pipe connection, similar to those of the transverse piezometers, was carried through the bulkhead, so that they might be connected to one side of the tube gauge. The reason for adopting this type of piezometer was that Mr. FitzGerald* records that he investigated the head, when measured in pipes, by a device of this sort in which the perforated pipe is laid upon the bottom of the large pipe, in comparison with the head as given by a piezometer consisting of a chamber surrounding the pipe and communicating with it by holes in a plane at right angles to the axis of the pipe, and he found that there was apparently no difference. So that on the strength of those experiments it was ventured to assume that this would probably give a correct reading of head at the point where it was wished to measure it, *i. e.*, at a point corresponding to Bazin's position, if there is such a thing as a correct reading of head. About 40 experiments were made and they showed, at the highest heads observed, a little over 3 ft., that there was a difference of 3 cm. in the head as read by the transverse piezometer and as read by the longitudinal piezometer, the new longitudinal piezometer giving a head 3 cm. higher than did the old up-stream transverse piezometer. As soon as these data were obtained a memorandum of it, sufficient to locate a correction curve, was furnished to the author, who, however, decided to reject the readings of the up-stream piezometer and to adopt those of the middle one, which was 10 ft. back from the weir. Now, having simultaneous observations on the middle piezometer and the up-stream piezometer, and having a series comparing the new piezometer with the up-stream piezometer taken through the later investigation, the new piezometer readings, assumed to be correct, were plotted as a straight line at an angle of 45° with the axes on which the heads were laid off, and with this the old transverse up-stream piezometer and the middle piezometer were plotted. The differences could not be detected at a head of 5 cm., $\frac{1}{6}$ ft., but at 3.3 ft. the difference between the new and the old upper piezometer was 3 cm., the latter being low at all

* "Flow of Water in a 48-in. Pipe," *Transactions*, Am. Soc. C. E., Vol. xxxv, p. 259.

TABLE No. 7.—Comparison of Simultaneous Discharge over Upper and Lower Standard Weirs by Bazin's Formula.

Experiment No.	Upper Weir. 16 ft. long, 13.13 ft. high. Obs. h. Transverse Piezometer. 27 ft. up stream.	Upper Weir. 16 ft. long, 13.13 ft. high. Cor. h Longitudinal Piezometer. 27 ft. up stream.	Upper Weir. 16 ft. long, 13.13 ft. high. Q = cubic meters per second.	Lower Weir. 6.56 ft. long, 5.2 ft. high. Obs. h. Flush Piezometer. 37 ft. up stream.	Lower Weir. 6.56 ft. long, 5.2 ft. high. Q = cubic meters per second.	Percentage of excess of lower over upper weir.
	Cm.	Cm.		Cm.		
13	12.075	12.28	0.3998	22.744	0.4053	+1.378
12	15.023	15.30	0.5499	27.855	0.5484	−0.278
6	18.069	18.39	0.7178	33.175	0.7124	−0.203
11	21.294	21.65	0.9133	39.419	0.9214	+0.886
7	23.759	24.16	1.0720	44.000	1.0915	+1.820
10	26.810	27.21	1.2785	49.699	1.3305	+4.067
1	29.658	30.16	1.4900	55.213	1.5456	+3.732
8	29.723	30.22	1.4945	55.128	1.5436	+3.283
9	37.437	37.90	2.0805	68.238	2.1440	+3.052
5	43.812	44.22	2.6280	80.566	2.7772	+5.677
4	58.110	59.00	4.4800	105.639	4.2645	−4.810
3	72.710	74.22	5.7300	130.286	5.9582	+3.981
2	79.565	81.69	6.6200	142.557	6.8881	+4.041

points. The curve of the middle piezometer started above the longitudinal one and reached a maximum difference at a head of about 2½ ft., where it was about 1 cm. high, and then dropped rapidly, appearing to cross the longitudinal piezometer at about 3.3 ft., after which it would read low. Now, of course, it cannot be affirmed that the new piezometer is a correct one to use with Bazin's formula, but, in view of the experiments of Mr. FitzGerald on the Rosemary Syphon, and in view of the comparison by the tube gauge of the transverse piezometer and the Francis piezometer it seems safe to infer that the new piezometer is the best one to use for the heads on this weir, or is as nearly correct as anything that we have. Furthermore, it was discovered that the head, as given by the new longitudinal piezometer, was much more steady and there was less vibration, less change and less pulsation than in the head as observed by the other. In order to complete the whole subject, an attempt was made to determine whether the manner of opening the gates, or the way in which the water was admitted to the chamber, would have any effect on the correction, and to that end all the head gates were opened a short distance, allowing the water to enter at the bottom of the chamber and flow over the weir. Then, at times, only two gates were opened, and they were opened wide, so that the water would enter from the bottom clear up to the middle of the height of the weir, but, with the maximum variation of flow which it was possible to obtain, there was no appreciable difference in the reading of the two piezometers, so that it is assumed that the correc-

Mr. Williams.

TABLE No. 8.—Experiments Upon Various Weirs for United States Board of Engineers on Deep Waterways at the Hydraulic Laboratory of Cornell University.

(1) Weir and No. of experiment.	Standard Weir, 16 Ft. Long, 13.13 Ft. High. h Read 27 Ft. Up Stream. (2) Obs. h. Cm.	(3) Cor. h. Cm.	(4) Discharge. Cu. Met.	Experimental Weir, 6.56 Ft. Long. h Read 38 Ft. Up Stream. (5) Obs. h. Cm.	(6) Cor. h. Cm.	(7) Description.
C = 3–1.	86.802	89.50	7.6030	158.006	170.00	5 to 1 up-stream slope, 8-in. flat crest; height of weir, 4.91 ft.
2.	59.205	60.10	4.1650	105.688	110.00	
3.	29.061	29.50	1.4420	51.841	52.42	
D = 4–1.	87.478	90.23	7.7040	155.159	166.45	4 to 1 up-stream slope, 8-in. flat crest; height of weir, 4.91 ft.
2.	72.707	74.16	5.7200	129.889	137.58	
3.	58.210	59.10	4.0580	103.937	108.02	
4.	43.714	44.15	2.6210	78.126	80.12	
5.	29.449	29.90	1.4715	52.260	52.85	
6.	14.583	14.81	0.5227	27.587	27.82	
E = 5–1.	87.146	89.80	7.6470	137.816	146.70	3 to 1 up-stream slope, 8-in. flat crest; height of weir, 4.90 ft.
2.	73.47	74.95	5.8155	117.268	123.00	
3.	57.78	58.59	4.0030	95.72	98.80	
4.	42.114	42.48	2.4675	74.04	75.72?	
5.	30.342	30.76	1.5340	50.02	50.65	
G = 7–1.	87.514	90.28	7.7080	134.399	142.75	2 to 1 up-stream and 2 to 1 down-stream slopes, 8-in. flat crest; height of weir, 4.895 ft.
2.	72.700	74.16	5.7200	114.005	119.20	
3.	57.998	58.84	4.032	93.813	96.72	
4.	43.778	44.19	2.6240	72.802	74.50	
H = 8–1.	87.168	89.82	7.6500	135.558	144.00	Same as G with $\frac{1}{4}$-in. mesh $\frac{1}{16}$ in. thick, wire-cloth netting on up-stream slope.
2.	71.970	73.40	5.6320	115.086	120.50	
3.	57.854	58.66	4.0120	94.270	97.27	
4.	43.958	44.36	2.6420	72.636	74.35	
5.	28.804	29.20	1.4210	49.217	49.77	
I = 9–1.	87.350	89.95	7.6660	138.198	147.10	2 to 1 up stream and 5 to 1 down-stream slope, 4-in. flat crest; height of weir, 4.94 ft.
2.	72.307	73.70	5.6675	117.366	123.00	
3.	57.702	58.51	3.9975	96.392	99.62	
4.	44.050	44.45	2.6495	74.510	76.22	
5.	29.404	29.82	1.4660	50.418	51.00	
J = 10–1.	88.406	91.19	7.8220	143.95	153.82	Vertical faces, 2.62-ft. flat crest; height of weir, 4.57 ft.
2.	73.626	75.05	5.8260	125.895	132.94	
3.	59.154	60.05	4.1610	106.66	110.98	
4.	45.168	45.58	2.7559	85.57	87.99	
5.	30.090	30.48	1.5115	60.676	61.70	
K = 11–1.	88.412	91.22	7.8300	140.010	149.15	Same as J, with 4-in. radius quarter-round added to up-stream corner.
2.	74.006	75.61	5.8930	121.39	127.70	
3.	58.592	59.48	4.0981	101.751	105.60	
4.	44.270	44.69	2.6690	80.511	82.52	
5.	30.540	30.90	1.5435	57.895	58.80	
L = 12–1.	72.662	74.10	5.7150	144.640	154.55	Vertical faces, 6.56-ft. flat crest; height of weir, 4.56 ft.
2.	57.89	58.71	4.0180	120.580	126.80	
3.	43.816	44.23	2.6300	93.760	96.75	
4.	29.40	29.80	1.4645	64.938	66.30	
M = 13–1.	72.632	74.08	5.7100	136.098	144.70	Same as L, modified as K.
2.	58.23	59.09	4.0550	111.67	116.60	
3.	43.71	44.16	2.6220	86.019	88.52	
4.	29.396	29.80	1.4645	59.000	60.02	
5.	14.982	15.21	0.5445	30.535	30.80	
N = 14–1.	87.22	89.90	7.6600	146.831	157.05	Rexford Flats Model; height of weir, 4.53 ft.
2.	72.59	74.00	5.7010	124.720	131.60	
3.	58.018	58.90	4.0378	101.567	105.40	
4.	43.722	44.18	2.6230	78.314	80.25	
5.	29.156	29.57	1.4470	53.549	54.25	
6.	14.442	14.66	0.5150	27.812	28.05	

TABLE No. 8—(*Continued*). Mr. Williams.

(1) Weir and No. of experiment.	Standard Weir, 16 Ft. Long, 13.13 Ft. High.			Experimental Weir, 6.56 Ft. Long.		
	(2) Obs. *h.* Cm.	(3) Cor. *h.* Cm.	(4) Discharge. Cu. Met.	(5) Obs. *h.* Cm.	(6) Cor. *h.* Cm.	(7) Description.
O = 15—1.	73.262	74.75	5.7910	122.953	129.55	Same as *N*, with rounded corner as in *K* and *M*.
2.	58.393	59.24	4.0745	101 191	105.00	
3.	43.555	44.00	2.6070	77.557	79.52	
4.	29.618	30.10	1.4848	53.041	53.65	
P = 16—1.	72.63	74.10	5.7120	12 .582	126.75	Little Falls Model. 3½ to 1 up-stream slope; height of weir, 4.57 ft.
2.	58.31	59.18	4.0660	98.957	102.55	
3.	43.56	44.00	2.6075	76.240	78.02	
4.	29.05	29.50	1.4425	51.514	52.00	
Q = 17—1.	72.878	74.40	5.7485	121.04	127.30	Little Falls Model; 3 to 4 up-stream slope; height of weir, 4.57 ft.
2.	58.326	59.22	4.0725	99.763	103.32	
3.	44.29	44.72	2.6730	76.552	78.39	
4.	29.034	29.50	1.4420	51.019	51.57	
5.	18.004	18.35	0.7140	32.038	32 40	
R = 18—1.	22.900	23.30	1.0170		38.523	Indian Lake Model; height of weir, 4.65 ft.
2.	86.074	88.60	7.4835		149.362	
3.	71.992	73.40	5.6325	Cor-	125.693	
4.	57.536	58.38	3.9810	rect	100.766	
5.	43.314	43.75	2.5850	head read	75.427	
S = 19—1.	14.338	14.59	0.5118	on	27.04	Submerged section, round crest, 1 to 1 up-stream slope; height of weir, 5.28 ft.
2.	28.435	28.85	1.3950	flush	51.36	
3.	85.786	88.36	7.4530	piezo-	142.128	
4.	70.915	72.36	5.5140	meter.	119.442	
5.	56.464	57.30	3.8710		97.858	
6.	43.812	44.38	2.6405		77.246	
7.	28.982	29.38	1.4330		53.42	

tions which were determined are probably as reliable as could be determined with the apparatus used, in which the heads were read in divisions of about 1 mm. Now, applying these corrections to Experiments Nos. 20 and 21, and applying Bazin's formula without the correction for velocity of approach, it appears that for heads on the standard weir running up to 23 cm., which means a head on the lower weir of about 1½ ft., that is, so long as the head on the lower weir was within the range of Bazin's investigations, the difference in the discharge, as given by the two weirs, is less than 2 per cent. In fact, for heads on the lower weir from a little less than 1 ft. up to about 1⅓ ft., the difference is less than 1 per cent. But, as soon as the head gets above that point the discharges depart very rapidly, the lower weir showing the higher discharge; and the variation ranges from 3% up to nearly 6 per cent.

In view of the conditions existing during the experiments it does not seem possible that the flow into the canal between the two weirs could have exceeded the leakage from it at the lower gates. In other words, it appears clearly impossible that more water passed over the lower than over the upper weir on this account: and yet, according to

Mr. Williams. the observations and formula, there is an excess of about 4% in the discharge of the lower weir, to be accounted for at the high heads. It is to be noted, however, that the observations at high heads were taken in general from high to low, and as the gauges were only read to 2-mm. divisions, it is quite possible that the canal surface may have been falling when it appeared to the observer to be stationary, and hence a somewhat greater discharge have passed the lower than the upper weir.

A portion of the excess possibly may be accounted for, that is, about 0.6 of 1%, by the narrowing of the notch of the lower weir due to the pressure of the water in the canal sides. It was contracted toward its top, slightly. The width at the top was not measured accurately. But such measurements as were made went to show that about 2 ft. from the top of the weir it was contracted nearly ½ in. This would make a difference of about 0.6 of 1% in the discharge at the higher heads. But that falls far short of accounting for all the difference. Mr. Wisner, in his discussion, has suggested that the effect of the side walls may very properly be considered to have something to do with this condition, and the extreme roughness of the sheet as it passed over the lower weir at the high heads will probably explain what remains.

In a later investigation it happened that there was obtained, incidentally, some notion of the effect of such roughness. There happened a repetition of two experiments in which the experimental weir remained in the same condition, while above the standard weir there were set additional baffles, between the two experiments, so that in the second case the water approached the weir much more smoothly than in the former. It appeared that with such roughness as existed in the standard weir, with heads of about 1½ ft. before smoothing, there was about ½ cm. more head required to deliver the same quantity of water than with the smoother approach. That is to say, if the water approaches the weir with high commotion a higher head will be required to discharge a given quantity than when the approach is smooth. The commotion at the standard weir, in the later experiments, was not to be compared with the commotion at the lower weir in the case of the high heads, in the earlier investigations. In the latter, and next to the wall, there was a roll, then came three crests and depressions, the bottom of the depressions being sometimes nearly 6 ins. below the crests. It seems, therefore, that when a proper correction is made for the effects of the roughness, the two weirs would come quite closely together. Of course, the reduction of the observations is simply applying Bazin's formula to the two weirs, *i. e.*, computing a discharge for weirs of that height according to his formula without any further corrections whatever. At the time these investigations were begun it was said by the Board of Engineers on Deep Waterways that if we could give them results within 6% of accuracy they would be abundantly satisfied.

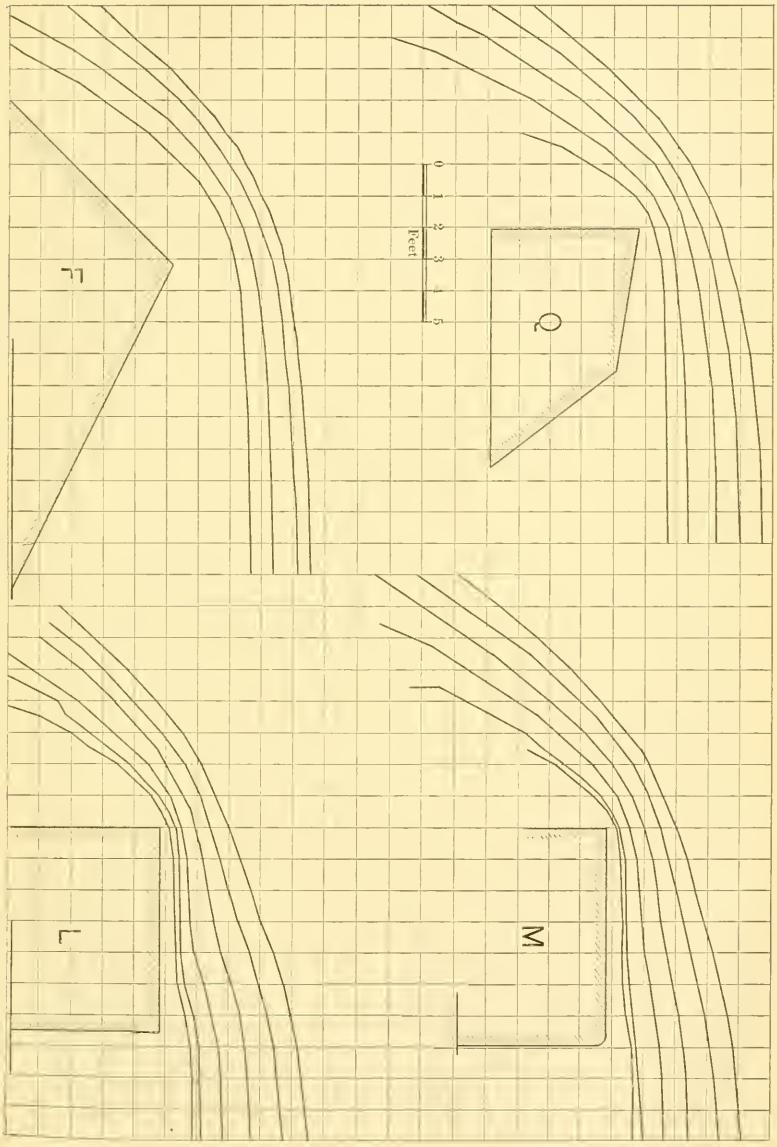


FIG. 21.

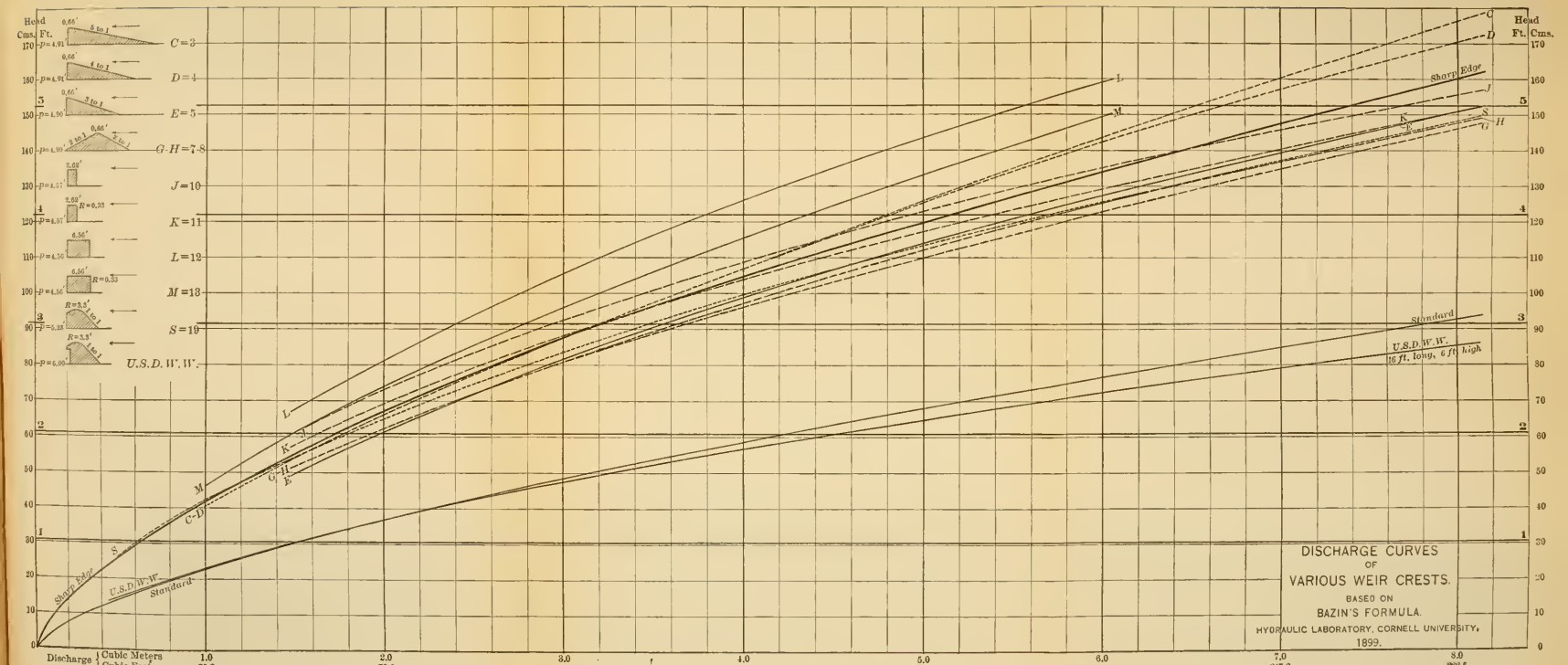
Mr. Williams. In the speaker's opinion, the results come within that range. He would not claim more. The Hydraulic Laboratory staff has performed experiments since that time, in connection with the Croton Water-shed investigations, for John R. Freeman, M. Am. Soc. C. E., which come far nearer to accuracy than 6%; but, so far as those which are given in the paper are concerned, it is very questionable if they can be depended upon within less than 6 per cent. Now, it appears to the speaker—it may be a notion in which he is peculiar—that in presenting the results of an investigation of this kind to this Society, in putting the observations upon record, forever as it were, it is most proper to present them first as nearly as possible as they were taken, to keep quite distinct the data which are facts and the data which are conclusions, to present the experiments as they were made with as little reduction as possible, so that in the future the investigator may determine for himself, in the light of such new knowledge as he may then have, just what reliability is to be put upon the observations, and what lessons are to be drawn from them.

The speaker would criticise the author for having presented here a paper in which practically all is reduction, and there is no getting behind his returns, whatever we may discover in the future as to the flow over weirs. So far as the data in this paper are concerned, there is little that we can go back to and make a rigid comparison with. That which has been presented is deduced from a computed discharge of the standard weir, which has been shown to be fundamentally in error. It then follows that the whole array of coefficients and coefficient curves on pages 267 to 284, inclusive, are similarly in error and therefore correspondingly reduced in value. This error probably ranges from zero to 3 per cent.

On Fig. 4 there is the following note: "The correction for Velocity Head $\left(\frac{v^2}{2g}\right)$ as used in Reducing the Experiments is in effect Equivalent to $h_v = 1.33\left(\frac{v^2}{2g}\right)$ for position of Piezometer 6.0 Ft. Back of Weir." Upon what authority this statement is made the speaker is unaware, but if there are any data upon which such a statement can be legitimately based it is to be regretted that the author did not give a reference thereto. So far as the Cornell experiments are concerned, there is nothing to lend support to such an assertion, and until some facts are brought to support it, it is only entitled to consideration as a rather positively expressed opinion, which, in the speaker's opinion, is contrary to fact, for the reason that we have no means of knowing how the head actually observed would compare with that which would have been observed at an opening in the side-wall, 6 ft. up stream from the crest.

In order that the results of this investigation may be properly on

PLATE XV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XLIV, No. 884.
WILLIAMS ON FLOW OF WATER OVER DAMS.



DISCHARGE CURVES
OF
VARIOUS WEIR CRESTS.
BASED ON
BAZIN'S FORMULA.
HYDRAULIC LABORATORY, CORNELL UNIVERSITY,
1899.

record, Table No. 8 is presented, in which Column 1 gives the number of the experiment in its proper series; Column 2 gives the observed head on the standard weir by the transverse piezometer 27 ft. up stream from the weir; Column 3, this head corrected to that read by the longitudinal piezometer 27 ft. up stream from the weir and 6 ft. above the bottom of the channel of approach; Column 4, the discharge per second, in cubic meters, by Bazin's formula, Mr. Williams.

$$Q = \left[0.405 + \frac{0.003}{h}\right]\left[1 + 0.55\left(\frac{h}{p+h}\right)^2\right] l\, h\, \sqrt{2\,g\,h},$$

where p = the height of the weir = 4.002 m. = 13.13 ft., and l = length of crest = 4.8768 m. = 16 ft.; Column 5 shows the head simultaneously observed upon the experimental weir at the up-stream transverse piezometer; and Column 6 this head reduced to the flush piezometer, or the head observed upon the flush piezometer in the bottom of the channel, 37 ft. up stream from the weir in the case of the experimental sharp edge, and 38 ft. in the cases of the other experimental weirs. All heads given are the means of those observed during the time of the experiment. These heads have been recomputed from the original field notes.

Series *A*, *B* and *F* (Nos. 1, 2 and 6 of the author), have been omitted, the results being too questionable to warrant insertion with the others, and *E* (author's No. 5) is considered as quite possibly inaccurate. The crests of the experimental weirs were approximately 2 m. = 6.56 ft. long.

While the absolute values determined by this investigation may be considerably astray, because of uncertainty, within at least 3% of the quantity of water passing the standard weir, the relative discharges of the several experimental weirs are of great interest, and on Plate XV the discharge curves of several of the types are shown, these curves being based upon that of the 16-ft. standard weir computed by Bazin's formula. These weirs were all of approximately the same height, the range being from 4.6 to 5.3 ft., so that from the plate one may readily see the effects upon the discharge caused by crests of various forms.

It will be noted that the discharge curve of the experimental sharp-edged weir divides the upper group of curves about in halves, those weirs whose curves fall above it giving a less discharge for a given head than does the sharp-edged weir.

One very interesting point is the behavior of broad flat crests. As seen by *L*, they give, at the lower heads, much less discharge than the standard, but, as is shown by *J*, and already pointed out by Messrs. Fteley and Stearns, and by Bazin, when the head reaches a point at which the sheet jumps from the up-stream edge clear or nearly clear of the down-stream corner, and the space between the sheet and crest becomes filled with eddying water, the discharge is very notably increased; so much so in the case of *J* that it exceeds that of the sharp-

Mr. Williams.

edged weir at 4.5 ft. head. The curves *M* and *K* show the increase of discharge due to building on a 4-in. radius, quarter round, to the up-stream corner of *L* and *J*; this rounded edge adding over 11% to the discharge at a 4-ft. head in both cases.

The effects of long and short back or up-stream slopes are shown by the curves *C*, *D*, *E*, *G* and *S*. *C* being 5 to 1; *D*, 4 to 1; *E*, 3 to 1; *G*, 2 to 1, with a 2 to 1 down-stream slope added; and *S*, 1 to 1 with a $3\frac{1}{3}$-ft. radius round crest. As stated by Bazin, when the inclination of the up-stream face of the weir is such as to form an obtuse angle with the bottom of the channel of approach, the tendency is to suppress the contraction of the sheet as it goes over the crest, and thereby increase the discharge, but if the up-stream slope be too gradual, the frictional resistances along it may be sufficient to counteract the gain in discharge from suppression of contraction. This appears to be the case with *C* and *D*, and, at low heads, with *S*, when the curved crest partakes of the nature of a long slope. At higher heads the 1 to 1 back-slope becomes effective and the discharge increases above that of the standard weir. From the upper curves, the weir *G* appears to have the maximum discharging capacity, but this seems to be in part due to the fact that the entry of air under the discharging sheet was restricted with it, but not with the others. The later experiments upon the United States Deep Waterways Section, 16 ft. long and shown with the 16-ft. standard weir in the two lower curves of the plate, wherein air was not admitted under the sheet, gives very nearly the same discharge at 3 ft. head, as did the weir *G*, both giving over 11% more than the sharp edge. The difference between curves *G* and *H* shows the effect of adding wire-cloth to the up-stream face of the weir *G*. Some later experiments indicate that the difference of discharge between a crest of dressed pine and one as rough as $\frac{1}{4}$-in. mesh wire-cloth will hardly amount to 3 per cent.

During this investigation, at the suggestion of Mr. George Y. Wisner, the side of the flume at the lower weir was marked off into squares, which were lettered and numbered so that the line of the surface of the approaching and discharging water upon the side of the flume could be read probably within 0.05 to 0.10 ft. During all but the first two experiments these squares were read, and from these readings eight of the most characteristic profiles of the surface curves, shown in Figs. 21 and 22, have been plotted.

Comparing *L* and *M*, the effect of the rounded corner is readily seen in the reduction of contraction at the up-stream corner, particularly at the lower heads. The influence of the back slopes is seen in comparing *D*, *F*, *Q* and *S*. It is to be regretted that the readings were not continued up stream to the beginning of the surface curvature, which, in some cases, was lost in the rapid at the throat of the flume, 48 ft. from the crests. From some of the experiments upon the effect of

Mr. Williams.

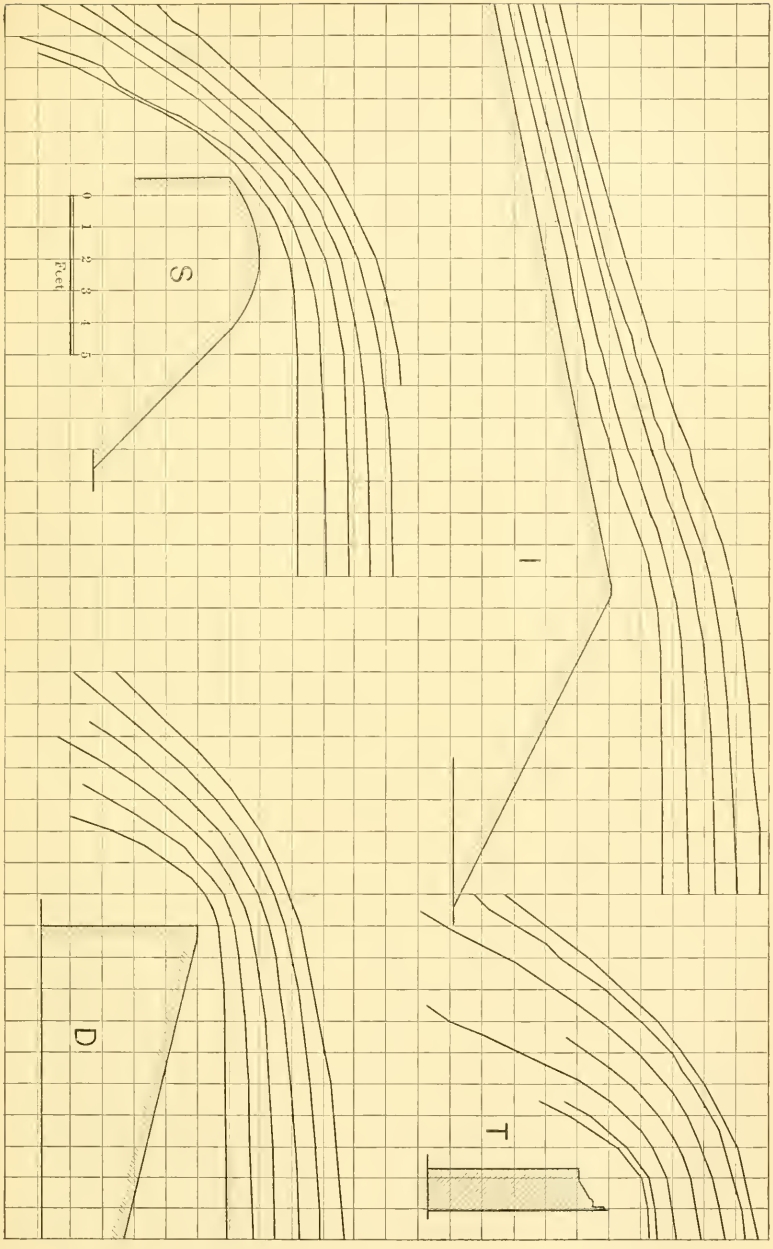


Fig. 22.

Mr. Williams.

contractions in pipes it seems very probable that this contraction may have seriously affected the discharge, and in future similar experiments it would seem well to remove it much farther from the weir or nullify its effect with baffles.

The weir *I* has a peculiar discharge curve. At low heads the flow is chiefly influenced by the 2 to 1 up-stream slope giving a high discharge, but as the head increases a point is reached where apparently the slope of the apron, 5 to 1, is not sufficient to maintain the velocity necessary to free the crest, and the discharge decreases relatively to that of a sharp-crested weir, the whole weir partaking, apparently, of the nature of a broad flat crest.

For the form of crest represented in Fig. 8, the Croton experiments, upon a large-sized model quite similar to this, indicate that the discharge partakes of the nature of that of the flat crest under high heads, the water between the crests of the old and new dam reducing the friction across the top, and thus producing or permitting a discharge slightly greater than that of a sharp edge, rather than giving one 20% less, as assumed by the author by comparison with the observation on crest *L*. This point can in no way be considered as a reflection upon the judgment of the author, as these data were not available at the time he made his estimates, and the matter is only presented to indicate how far one may go astray on these questions, unless exact information has been obtained on the specific form considered.

One of the most important facts brought out in the past year's investigations in the Cornell Hydraulic Laboratory has been the formation of a vacuum more or less perfect behind the falling sheet when air is not freely admitted. With a weir 6 ft. high, the United States Deep Waterways Section, a head of 1.5 ft. has been observed to raise water behind the sheet to a height of 2 ft. above the level of the lower pool, and with a weir 8 ft. high and a 2-ft. head the water behind the sheet has stood 5 ft. above the level of the lower pool. The bottom boards of the plank aprons have been torn off frequently, apparently by the suction of the falling sheet at the toe of the dam. The possibility of a vacuum upon the down-stream face of a dam has, so far as the speaker is aware, never been considered in the design of such structures, but the pulling off of the granite facing on the down-stream side of the Austin Dam, while that at the crest remained practically intact, and other instances of similar phenomena that have been reported, seem to indicate that there may have been a very decided suction there on the occasion of its failure. This teaches that in the design of spillways, the practice of conforming them to the curve of the sheet, in order to obtain a smooth and compact overfall, should be reversed, and every precaution taken to prevent the sheet reaching the foot of the dam in a compact mass.

In conclusion, the speaker would acknowledge his great indebted-

ness for the very valuable services of his colleague, Mr. W. E. Mott, in the reduction and preparation of the data herein referred to and presented, and also in assisting in observing, under very trying conditions, during many of the later experiments. Mr. Williams.

E. KUICHLING, M. Am. Soc. C. E.—This paper may fairly be regarded as a useful contribution to the literature of hydraulics, as it gives us a translation of Bazin's summary of his recent experiments on the flow of water over weirs and dams of various section, along with data obtained at the Hydraulic Laboratory of Cornell University relating to the flow over a number of dams, of both larger and different section from those used by Bazin, and under much greater heads. The method of conducting the experiments at said laboratory appears to have been similar in general to that which was followed by Bazin, except that no direct volumetric determinations of the discharge were made, as no facilities for this purpose are yet available there. On this account the calibration of the large standard reference weir could be done only by assuming the accuracy of some one of the various existing weir formulas; and, as Bazin's formula was not only the most recent one, but also purported to consider fully the influence of the height p of the standard weir above the bottom of the channel of approach, it was accordingly selected by the author as best representing the law of discharge. Mr. Kuichling.

Owing to the various algebraic errors in the paper as originally submitted, it becomes proper to state here that the correct form of Bazin's preferred formula, adapted to measures in English feet, for the flow over a sharp-crested vertical weir with end contractions suppressed, is:

$Q = m\, l\, h \sqrt{2\, g\, h}$, where the coefficient

$$m = \left(0.405 + \frac{0.00984}{h}\right)\left[1 + 0.55\left(\frac{h}{p+h}\right)^2\right],$$

and h is the observed head or depth of the water above the crest, measured at a distance of 5 m., or 16.4 ft., above the weir, while p is the height of the horizontal weir crest above the bottom of the channel, and l is the length of said crest or the width of the channel of approach, which is assumed to be of uniform section, with vertical sides and horizontal bottom. An apparatus of this description is called a standard weir when provision is made for the free admission of air under the falling sheet of water.

It should also be remembered that this formula was established from comparative experiments with standard weirs of different length and height, within a rather limited range and with comparatively low heads. Bazin's largest weir was only 6.56 ft. long and 3.72 ft. high, on which a maximum head of about 1.0 ft. was used, on account of restricted volumetric capacity. For heads from 1.00 to 1.32 ft. it was necessary for him to reduce the length of this weir one-half, or to

Mr Kuichling. 3.28 ft.; and for heads from 1.32 to 1.77 ft., the latter length was again reduced one-half, or to 1.64 ft., while the height was reduced to 3.30 ft. In reference to the experiments made with the last-mentioned weir, Bazin states that as it was possible for the retardation by friction against the side walls to become appreciable in so small a weir or channel, the observations made therewith were used by him only to deduce the law of increase of the coefficient m for heads greater than 1.32 ft., whereas the more numerous and precise results furnished by the two longer weirs served as the basis for determining the discharge of the standard weir. The computed values of the coefficient m for said longer weirs under the same heads up to 1.00 ft. were found to agree quite closely, whence Bazin inferred that for moderate values of h the law of the discharge $q = \frac{Q}{l}$ per unit of length of the weir was not materially affected by the length l, provided that the same was more than 3.0 ft., and that the curve obtained by plotting the values of m as ordinates to the corresponding values of h as abscissas could fairly be extended from $h = 1.00$ ft. to $h = 2.0$ ft., governed somewhat in shape by the curve for m given by the shortest weir.

Furthermore, to find the effect of varying the height p of a standard weir, Bazin next compared the heads h for known discharges Q over his longest weir, having $l = 6.56$ ft. and $p = 3.72$ ft., with those which were observed for the same discharges over four secondary standard weirs of the same length l, but having heights of 2.46, 1.64, 1.15 and 0.78 ft.; and he also computed and plotted the values of m for each series of experiments corresponding to the different values of h. The latter, however, did not exceed $h = 1.48$ ft. on the standard reference weir having $p = 3.72$ ft. Within these limits of head and height, and the same length l, it was found that the values of the coefficient m derived from experiment did not vary more than 1% from those given by the aforesaid expression :

$$m = \left\{ \left(0.405 + \frac{0.00984}{h}\right) \left[1 + 0.55 \left(\frac{h}{p+h}\right)^2\right] \right\}$$

except in the case of the lowest weir, were the value of m for small heads given by the formula is about 2% more than found by experiment, although such excess gradually reduces to zero as the head h increases to 1.30 ft.

Such is the foundation of Bazin's formula for the discharge over standard weirs with suppressed end contractions and heads h measured 16.4 ft. above the crest, and the question now arises whether it can be applied without material error, either to the large standard weir at the Hydraulic Laboratory of Cornell University, where the length $l = 16.0$ ft., the height $p = 13.13$ ft., and the heads h, up to a maximum of 3.0 ft., were measured 10 ft. above the crest, or to

the author's secondary standard weir at the same place, where the length $l = 6.53$ ft., the height $p = 5.26$ ft., and the heads h, up to a maximum of 4.874 ft., were measured 37 ft. above the crest. Mr. Kuichling.

In the absence of direct experimental evidence, little further can be said on this subject except to quote the remark of the late Hamilton Smith, Jr., M. Am. Soc. C. E., made on page 169 of his classic treatise on hydraulics, in concluding his comparison of the values of the coefficient of discharge for weirs of different length and large heads, that "such speculations, though possibly ingenious and plausible, when tested with facts, generally prove to be very wide of the truth." While this remark was intended to apply to cases far beyond the limits of accurate experiment, it is, nevertheless, also pertinent in some degree where such limits have been considerably exceeded, as in the present case; and hence it will be prudent to suspend judgment about the accuracy of the large discharges until the figures are properly corroborated. Under these circumstances, accordingly it, appears admissible to indulge in a little speculation about Bazin's formula for the coefficient m.

In this formula both the observed head h and the height p of the weir above the bottom of the uniform channel of approach are taken into account, but the width of the channel or the length l of the weir is omitted for the reason already given. Now, while the retardation of the flow by friction on the sides of the channel in the vicinity of the weir may be negligible in the case of long weirs and small heads, or up to a certain limiting ratio $\frac{h}{l}$, it is evident from observations of the relative velocities of thick sheets of water that beyond such limit this lateral friction cannot be disregarded; and hence a rational formula of general applicability should make the value of the coefficient m dependent also upon the channel width or weir length l, thereby allowing the formula for the discharge over a standard weir to pass gradually with reducing value of the height p into that for a uniform channel. This condition is not embraced in Bazin's formula, as will be seen from the following consideration.

When the height p becomes very small relatively to the head h, or disappears entirely, the above formula for m becomes practically $m = 1.55\left(0.405 + \frac{0.00984}{h}\right) = a + \frac{b}{h}$, while that for the discharge Q becomes: $Q = m\ l\ h\ \sqrt{2\,g\,h} = \sqrt{2\,g}\left(a + \frac{b}{h}\right) l\,h\,\sqrt{h}$, which should obviously correspond with the formula for the flow in a uniform open channel. Taking for comparison in this case the Chezy formula on account of its simplicity, we will have for the same discharge: $Q = A\ v = A\ c\ \sqrt{r\ s}$; and by replacing the section A by its equiva-

Mr. Kutchling. lent $l\,h$ in the nomenclature of the weir formula, and the hydraulic radius r by its equivalent $\frac{l\,h}{l+2\,h}$, there follows:

$$Q = c\sqrt{\frac{l\,s}{l+2\,h}}\ .\ l\,h\,\sqrt{h}$$

By equating these two values of Q, which are manifestly alike, we obtain: $\sqrt{2\,g}\left(a+\frac{b}{h}\right) = c\sqrt{\frac{l\,s}{l+2\,h}}$, which is an entirely incongruous result, being true only for particular values of l, h and s when the value of c is given. The inference is, therefore, justifiable that Bazin's formula can only give correct results in the neighborhood of the experimental limits from which it was derived, and that discordant results will probably be found if it is applied to conditions considerably beyond said limits.

TABLE No. 9.

For observed heads on weir of:	h =	1.00	2.00	3.00 ft.
1. Author's values of coefficient m for the experimental standard weir 6.56 ft long and 5.26 ft. high, are, as per page 295.....	m =	0.4174	0.4106	0.4094
2. Values of m computed by writer from Bazin's formula for same weir, are.....	m =	0.4207	0.4270	0.4379
3. Values of m deduced by writer from Bazin's table in Annales des Ponts et Chaussées, 1888, page 446, for a standard weir 1.60 m., or 5.26 ft. high, are.....	m =	0.4232	0.4252	Beyond limit of Table.
4. Values of m computed by writer from Bazin's formula with p = 13.13 ft., corresponding to large standard weir at Cornell University Laboratory, are.....	m =	0.4160	0.4139	0.4160

It may also be mentioned that Bazin's expression for m attains a minimum value for a particular value of h with each value of p. By applying the usual mathematical process for maxima and minima to said expression it will be found that for Bazin's standard weir with $p = 3.723$ ft., m will become a minimum $= 0.4245$ for $h = 0.814$ ft.; for the author's experimental standard weir at Cornell University, with $p = 5.26$ ft. m will become a minimum $= 0.4207$ for $h = 1.003$ ft.; and for the large standard weir at the same place m will become a minimum $= 0.4137$ for $h = 1.765$ ft. On comparing these latter values of m, as well as a few other computed values of m for $h =$ 1.0, 2.0 and 3.0 ft., with those given by the author in Fig. 2, a considerable disagreement will be noticed, whence a numerical error has probably been made in the author's computations. The text also indicates that the said values of m in Fig. 2 refer to the experimental standard weir with $l = 6.53$ ft. and $h = 5.26$ ft.: but on exam-

ining the diagram of discharge of the large standard weir with $l =$ 16.00 ft. and $p = 13.13$ ft., in Fig. 4, it will be found that the same values of m, or $\frac{Q}{l}$, were used as given in Fig. 2 for the smaller weir; and if such be the case another slight error has been made in the paper. The discrepancies in the values of m just mentioned are exhibited in Table No. 9. Mr. Kuichling.

The use of the author's experimental sharp-crested weir, 6.53 ft. long and 5.26 ft high, has not been made clear in the text. Apparently, it was built for the purpose of calibrating the large standard weir, 16.00 ft. long and 13.13 ft. high, but as the author states explicitly on page 292 that "for the reduction of Experiments Nos. 20 and 21 (referring to said two weirs) a discharge curve has been computed for the upper (large) weir for heads up to 0.6 m. (1.969 ft.), using Bazin's formula," it is not easy to discover its significance unless to find therefrom the comparative values of m for large heads measured at a distance of 37 ft. above its crest. In this event the experimental data would have been of much interest, as they would have established in some degree the truth or error of Bazin's formula when applied to high heads, and it is to be hoped that the author will supplement his paper with such data.

Another point of interest is the matter of the proper distance above the weir or dam at which the head h should be measured, as well as the details of such measurement; but as this part of the subject has been fully considered in the discussion by Gardner S. Williams, M. Am. Soc. C. E., further reference thereto may here be omitted.

E. A. Fuertes, M. Am. Soc. C. E.—The remarks of Mr. Williams, upon the lack of methods for performing volumetric measurements at the Hydraulic Laboratory of Cornell University, are pertinent and justifiable; but it must be borne in mind that the laboratory is as yet unfinished. There are, indeed, methods available, on the spot, for more or less accurate and direct volume measurements, but not of the nature intended to be made in the future, and for which, fortunately, topographical conditions lend themselves admirably to measure almost any desired volume of water. So far, this laboratory is a huge engine, with possibilities. Its final suitableness is a question of time and money. The speaker wishes to impress upon engineers the fact that such a laboratory exists; that its resources can be extended unlimitedly, and that one of the methods which will bring us quickly to the desirable conditions of which Mr. Kuichling speaks is to enlist the good offices, the good-will and the aid of the hydraulicians of this country to send us problems to solve, either under our own direction or under the direction of such visiting engineers as may desire to superintend the work. Mr. Fuertes.

It seems advisable to say, however, that while Cornell University will do everything that may be possible with its personnel, equipment

Mr. Fuertes. and other facilities for the progress of hydraulic science, it seems reasonable that such expense as may be necessary and may not be covered by the facilities of the laboratory at the time should be defrayed by the party seeking information. Nothing will be stinted on the part of Cornell University.

Mr. Horton. Robert E. Horton, Esq. (by letter).—The results of the Cornell experiments and those of Bazin emphasize primarily the necessity of using a variable coefficient in the weir formula, whenever the flow over an irregular-crested dam is to be computed. To facilitate calculations of this character, and thereby make the results of these experiments more readily applicable in practice, the accompanying diagram, Plate XVI, has been prepared. Having selected the value of the coefficient C, required for the given head and section, its value is scaled off on the line of ordinates at the left of the diagram and traced horizontally to its intersection with the transversal corresponding to the head required; the projection of this point of intersection on the line of abscissas will show the flow over the dam in cubic feet per second for one lineal foot of crest, without lateral contractions.

The chief distinction between the experiments made at Cornell University and those of previous experimenters, including Bazin, lies in the fact that the former were made on "life size" dams, as it were, and extended through a much wider range of heads. This is illustrated by Table No. 10, which shows the superior limit of various experiments on flow over sharp-crested weirs.

TABLE No. 10.

Name of experimenter.	Date.	Limiting head used in experiments, in feet.	Length of crest of experimental weir, in feet.	Distance up stream from weir to point of measuring H, in feet.
Poncelet and Lesbros..	1828	0.682	0.66	11.48
Lesbros.....	1834	0.8009	0.6562	11.48
Francis.....	1852	1.62	9.995	6.00
Hamilton Smith, Jr...	1874–76	1.7327	2.586	7.6
Fteley and Stearns....	1878	1.6038	19	6.0
Bazin.....	1886	1.77	6.56	16.40
Cornell experiments...	1899	4.672	6.56	38.0

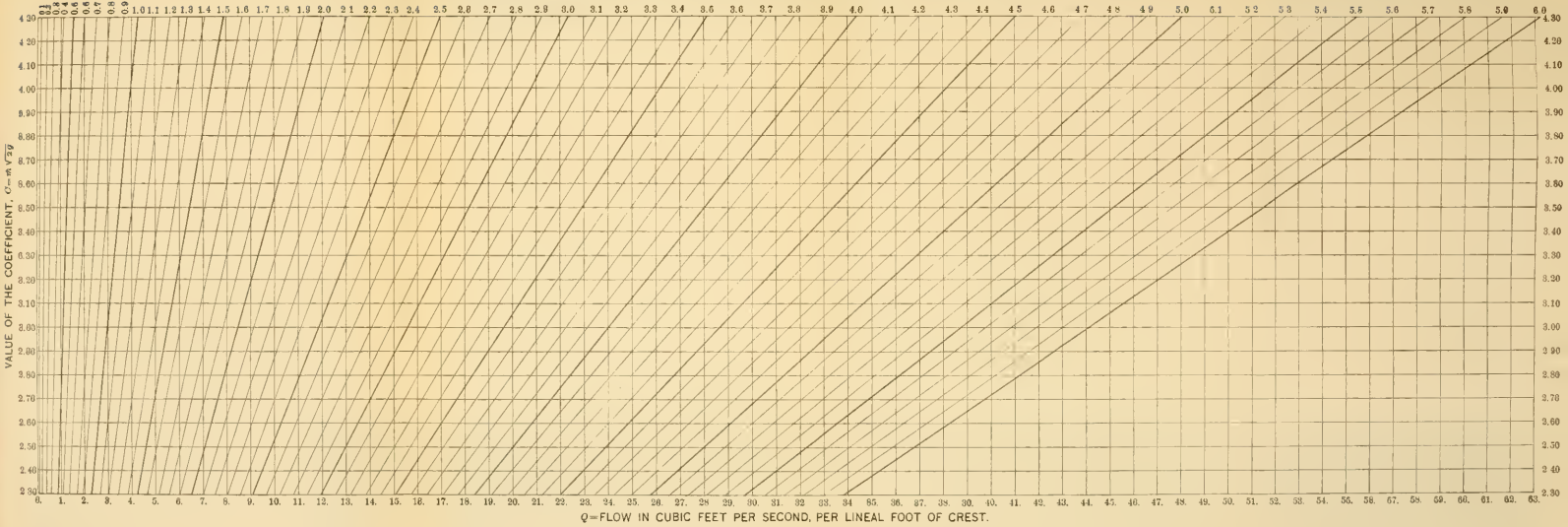
Owing to the limitations pointed out by the author, the Cornell experiments were less elaborate as regards minuteness of detail than were those of Bazin; as a rule, only four or five experimental coefficients were determined as a basis for plotting the mean coefficient curves. Bazin usually obtained twenty or more experimental coefficients, within a much narrower range of variation of head. The advantage of repeating the experiments for small increments of head will not be as

PLATE XVI.
TRANS. AM. SOC. CIV. ENGRS
VOL. XLIV, No. 884.
HORTON ON FLOW OF WATER OVER DAMS.

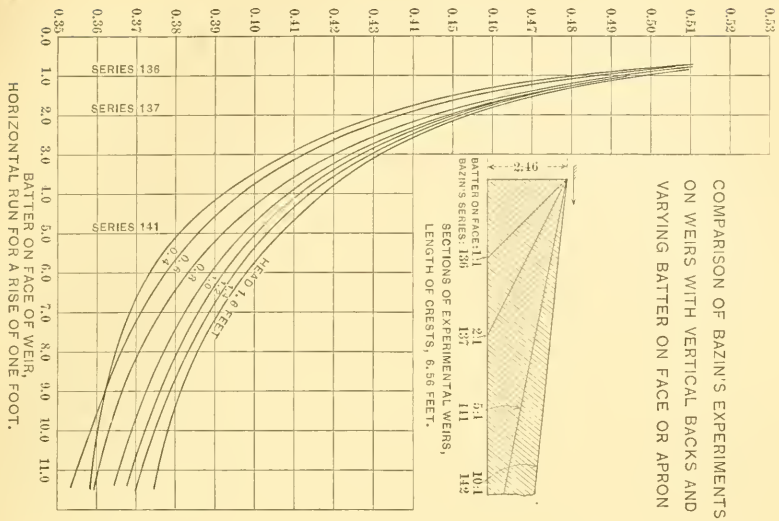
DIAGRAM SHOWING FLOW OVER DAMS,

$Q = CLH^{\frac{3}{2}}$

TRANSVERSALS REPRESENT DEPTH ON CREST OF WEIR IN FEET, MEASURED TO THE SURFACE OF STILL WATER.



Mr. Horton.



COMPARISON OF BAZIN'S EXPERIMENTS ON WEIRS WITH VERTICAL BACKS AND VARYING BATTER ON FACE OR APRON

Fig. 23.

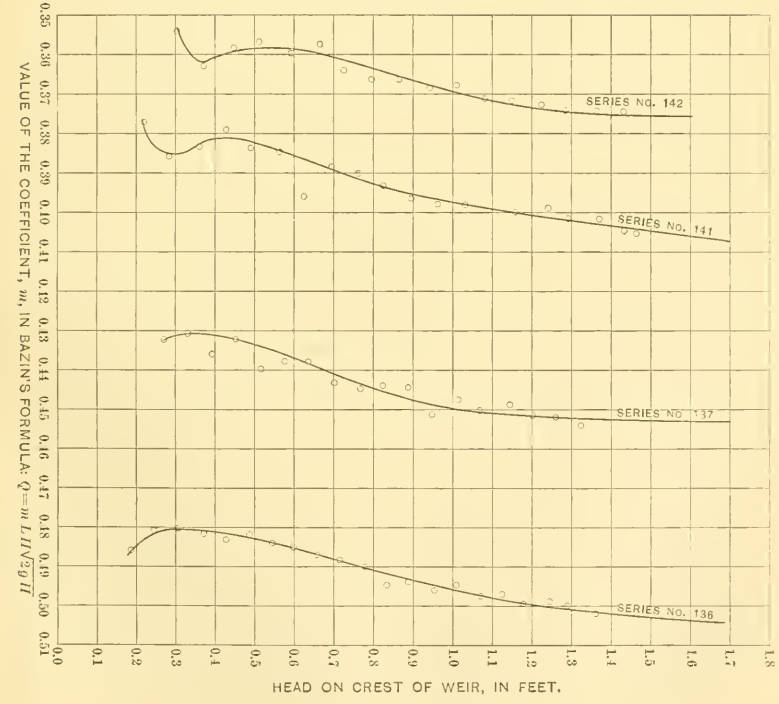


Fig. 24.

Mr. Horton. great, however, as might at first be imagined, in cases where the species of nappe does not change. In accordance with a well-known law of Least Squares, the precision of the mean coefficient curve will be proportional to the square root of the number of observations it represents. Quadrupling the number for each section would result in dividing the numerical factor for precision by two; moreover, the coefficient for each period in the Cornell University experiments is deduced from the mean of a relatively large number of observed heads on the weirs. This number, being more than twenty in each case, is so great that any moderate increase in the number would not have resulted in a proportional gain in precision. An examination of the coefficient curves will show that, as regards the individual coefficients, they appear no more discordant when plotted than do those of Bazin.

In the design of dams with depths sometimes as great as 12 ft. on their crests, there is a pressing need for data which will apply with a reasonable degree of accuracy. It is to be hoped that the Cornell experiments may be extended by the determination of additional coefficients for intermediate heads, and by experiments on a wider variety of dam sections than was possible at the time these experiments were made. Hitherto, engineers have generally accepted the standard sharp-crested weir as being the most reliable means of measuring relatively small volumes of flowing water. A free nappe, which is neither depressed, adherent, nor wetted underneath, represents the ideal condition, and might be termed a "Standard Nappe." This condition, as has been pointed out by Bazin, is very difficult of attainment. Most of the modifications to which the nappe is subjected under different conditions are phenomena belonging to comparatively low heads; for many, if not all, forms of weir sections, the nappe undergoes these various metamorphoses with heads not exceeding from 1 to 2 ft., and then settles down to a stable form which is retained for all higher heads. These facts point to the conclusion that weirs or dams are best adapted for the measurement of large streams of water forming depths of several feet on their crests.

Francis did not experiment with depths of flow exceeding 1.6 ft., although he computed discharge tables, by means of his formula, for heads up to 3 ft., and, as pointed out by the author, the constant coefficient, 3.33, has commonly been applied for all heads, however great. The formula of Bazin furnishes, perhaps, the most reliable means of determining the flow over low weirs with slight heads, and, in reducing the Cornell experiments, it has been used as a basis for determining the flow over the standard sharp-crested weir. Referring to Fig. 2, it will be seen that, for heads from 1 to 6 ft., the discharge coefficients deduced from the Cornell experiments Nos. 20 and 21, for a standard sharp-crested weir, remain nearly constant, varying only between the limits 3.350 and 3.284. The average value is 3.306, or

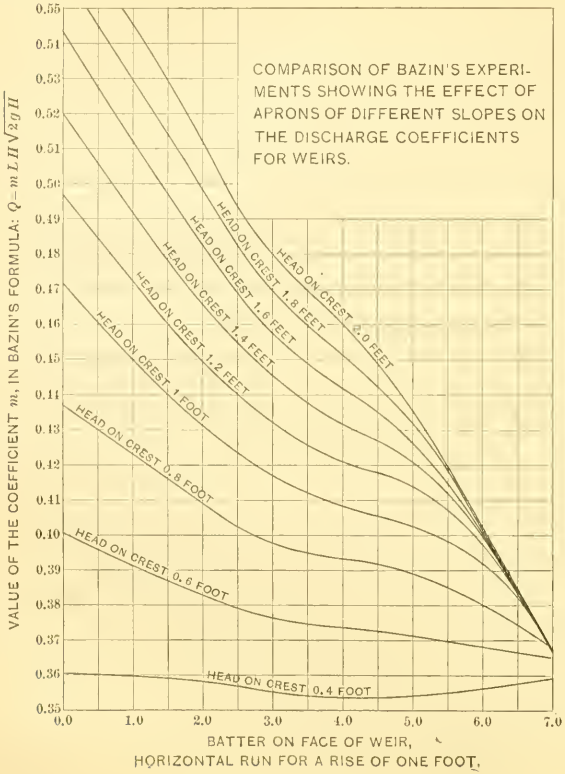

COMPARISON OF BAZIN'S EXPERIMENTS SHOWING THE EFFECT OF APRONS OF DIFFERENT SLOPES ON THE DISCHARGE COEFFICIENTS FOR WEIRS.
HEAD ON CREST 2.0 FEET
HEAD ON CREST 1.8 FEET
HEAD ON CREST 1.6 FEET
HEAD ON CREST 1.4 FEET
HEAD ON CREST 1.2 FEET
HEAD ON CREST 1 FOOT
HEAD ON CREST 0.8 FOOT
HEAD ON CREST 0.6 FOOT
HEAD ON CREST 0.4 FOOT
VALUE OF THE COEFFICIENT m, IN BAZIN'S FORMULA: $Q = m L H \sqrt{2gH}$
BATTER ON FACE OF WEIR, HORIZONTAL RUN FOR A RISE OF ONE FOOT.

FIG. 25.

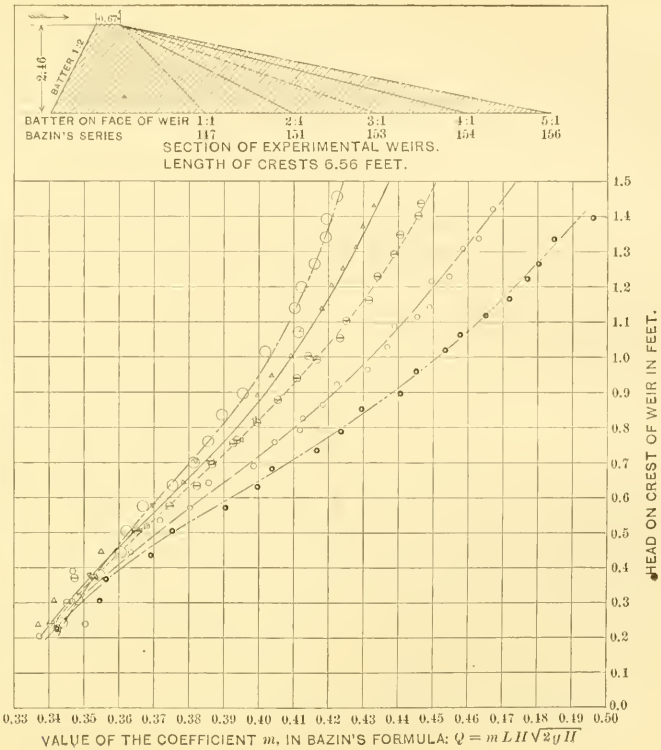

0.67
2.46
BATTER 1:2
BATTER ON FACE OF WEIR 1:1 2:1 3:1 4:1 5:1
BAZIN'S SERIES 147 151 153 154 156
SECTION OF EXPERIMENTAL WEIRS.
LENGTH OF CRESTS 6.56 FEET.
VALUE OF THE COEFFICIENT m, IN BAZIN'S FORMULA: $Q = m L H \sqrt{2gH}$
HEAD ON CREST OF WEIR IN FEET.

FIG. 26.

Mr. Horton. 0.7% less than the constant value 3.33, assumed by Francis, from a series of experimental coefficients varying from 3.31 to 3.35. Francis' formula, therefore, is practically verified for heads up to the limit of the Cornell experiments, and its reasonable use, for any ordinary head on a sharp-crested weir, appears to be justified.

It is greatly to be regretted that Bazin's experiments were not conducted on such a scale as to make his coefficients legitimately applicable to American mill-dams. This they certainly are not, and, while they may serve a useful purpose in measuring the flow of water in small modules, as in the partition of streams for irrigation purposes in the Western States, yet their chief interest to American engineers lies in the large number of facts which Bazin brought conspicuously before our attention, regarding the physical phenomena accompanying flow over weirs. The value of these results, qualitatively rather than quantitatively, can hardly be overestimated.

In this connection, one or two points may perhaps be brought out more strongly than has been done by Bazin in publishing his original researches. In order to show the effect of aprons of varying batter or slope, on the discharge coefficient, Bazin's Series, Nos. 136, 137, 141 and 142 have been selected. The experimental coefficients for each series have been plotted, and curves have been drawn, intended to fairly represent the experiments, as shown in Fig. 24. In each of these experiments the nappe appears to have adhered to the crest throughout all but the very lowest heads, for which there may have been a partial vacuum underneath the nappe, producing the short turn in the base of the first and second coefficient curves. In each case the coefficients increased gradually with the head, indicating that the effect of the friction of the long flat apron diminished in proportion as the head increased. This illustrates a general statement which may be made that, as the head increases, the influence of peculiarities in the form of the weir section, in modifying the discharge coefficient, becomes less and less significant, and, under excessively high heads, the coefficient curves for all cross-sections will tend to converge.

A series of coefficients for various heads, taken from Fig. 24, has been plotted in a slightly different form in Fig. 23, using the coefficients as ordinates, and the slope or batter on the down-stream face as abscissas. This brings prominently to our notice the greatly diminished discharging capacity of a dam with a flat apron, as well as the tendency of all the curves to become tangent to a common asymptote under high heads.

Comparing, in the same manner as before, Bazin's Series Nos. 147, 151, 153, 154 and 156, we obtain the series of coefficient curves shown in Fig. 26. The cross-sections for these series are precisely like those shown in Figs. 23 and 24, except that, instead of having a vertical

Mr. Horton.

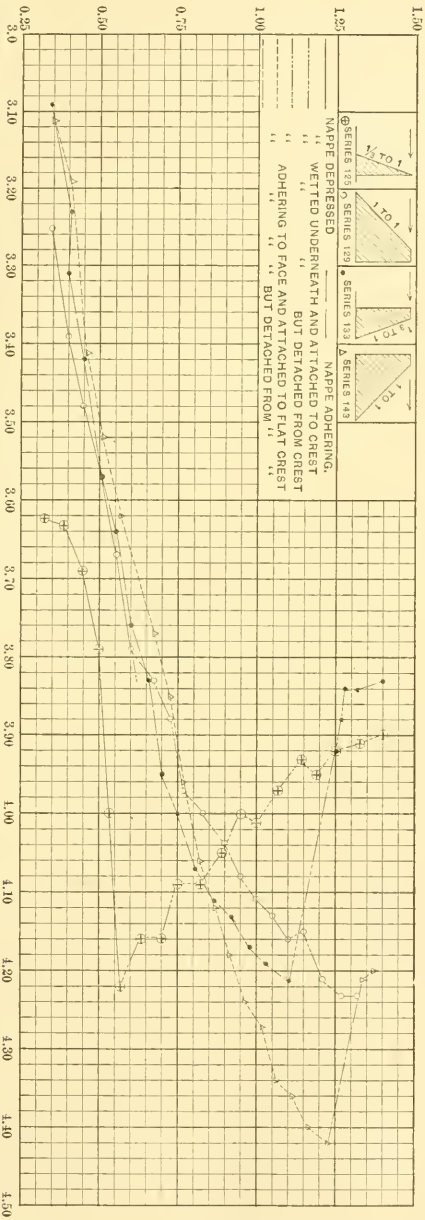


Fig. 27.

Mr. Horton. back, the up-stream face has a batter of 1 horizontal to 2 vertical, and a flat crest 0.67 ft. in width. The forms of the coefficient curves and the characteristics of discharge appear to be completely modified. Starting with nearly the same coefficients for all series at low heads, the coefficient curves diverge rapidly to the limit of the experiments. For low heads the influence of the apron is completely masked by the effect of the broad flat crest.

A number of similar comparisons might be made from Bazin's experiments, but enough has been given to show that no general formula for flow over a dam of irregular section, using the slopes of the faces and the breadth of the crest as independent variables, can be deduced; the coefficient curve for each combination must be determined separately, although the way in which any single element tends to modify the discharge may readily be pointed out.

In all the series of experiments above discussed, the nappe retained a sensibly stable form throughout the entire range of the experiment, being attached to the flat surface of the apron in each instance. The coefficient curves are continuous smooth lines. Where the nappe becomes depressed, detached or wetted underneath, during the progress of the experiment, the resulting coefficient curve, mathematically considered, will be a series of discontinuous arcs, possibly disconnected, and terminating abruptly in *points d'arret*, whenever the form of the nappe changes. Four series of coefficients, as deduced by Bazin, for weirs with variable nappes, are shown in Fig. 27. The condition of the nappe in each instance is indicated on the diagram. For dam sections like these the nappe may assume four distinct forms as regards its behavior with reference to the down-stream face of the weir, and, in addition, it may either adhere to, or be detached from, the crest. Either of these conditions may occur in combination with any of the four previously mentioned, so that by simple permutation we find that eight series of discharge coefficients are possible for a single form of section. On the other hand, there are many forms of weirs, such, for example, as that used in Cornell experiment No. 19, for which the nappe adheres to the crest and apron under all heads, and is subject to no modifications whatever. Owing to the permanence of the nappe form, such a section possesses some advantages over a sharp-crested weir for use as a standard section in making weir measurements.

Mr. Parmley. WALTER C. PARMLEY, M. Am. Soc. C. E. (by letter.)—This paper is a welcome addition to the literature on weir flow. About all that has been known of this subject up to the present time is what has been given to us in the work of Francis, Fteley and Stearns, and Bazin. Their experiments cover a rather narrow range of conditions, and hence the experiments carried out at Cornell make a substantial addition to our knowledge of the subject.

Mr. Parmley.

In the method of measuring the head upon the weir, Mr. Rafter has followed in general the processes used by the other experimenters, although in details they are varied. The height above the crest of the weir is not taken by direct observation upon the water surface, but by observations of the water surface in glass gauges at the side of the channel. As a consequence, tedious and involved corrections must be applied to the readings obtained from one set of piezometers in order to make them check with the results of the others. This method introduces uncertainty as to the proper place in the channel of approach of placing the inlet tubing, the manner of access to it, and the possibility of eddies vitiating the results. It is doubtful if holes bored in round tubing will give a correct record of the head, inasmuch

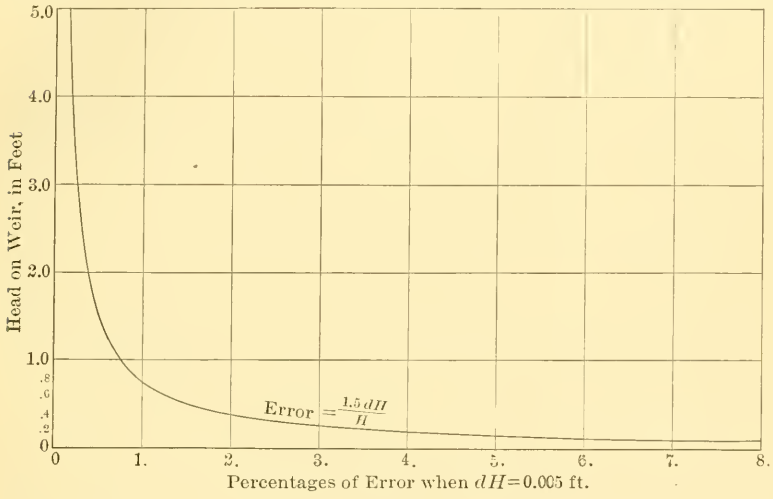


FIG. 28.

as the orifice is in a curved surface, when theory and experiment agree that the orifice should be at right angles to the motion of the water and through a plane surface parallel therewith. The experiments of Fteley and Stearns showed that the pressure near the up-stream side of the weir varied materially from the bottom to the top of the channel and with the distance from the weir. It is necessary, therefore, that the tubing enter the channel up-stream from this region of erratic pressures and, in a given case, it is not altogether certain just where these variations of pressure cease. The author does not explain whether or not any correction for capillary action in the piezometer tube was made, nor the amount, if such correction was made.

It seems to the writer that the method of Bazin is preferable. He provided recesses at the side of the channel where the actual surface

Mr. Parmley. of the water could be measured with a hook gauge. It undoubtedly has the advantage of avoiding the uncertainties mentioned. The author states that the readings of the piezometers were taken to 2 mm., that is, to within about 0.0066 ft. While an error of this amount is of no consequence when large heads are being measured, it becomes of considerable importance if the heads are small.

From his experience with many weirs under varying conditions, the writer has found the use of a steel tape graduated to hundredths of a foot very satisfactory. A sharp-pointed plumb-bob is attached to the end of the tape by means of a wire hook. The measurement to the surface of the water is taken from some well-defined point conveniently located above the water surface. These readings are made to thousandths, and in ordinary work can be made with certainty to within 0.005 ft., which is closer than the readings by the author's piezometers.

In order to illustrate the importance of accurate measurements when small heads are upon the weir and the relative unimportance of such accuracy for large heads, let us take the ordinary Francis formula, considering only a length of 1 ft. of weir.

$$Q = C\, H^{\frac{3}{2}}$$

Differentiate, and we have:

$$d\, Q = \tfrac{3}{2}\, C\, H^{\frac{1}{2}}\, d\, H$$

Now, the error of any gauging when $H + d\, H$ is taken as the head, instead of the true head H, will be the quantity dQ, and the ratio of the error to the true quantity Q, will be,

$$\frac{d\, Q}{Q} = \frac{3\, C\, H^{\frac{1}{2}}\, d\, H}{2\, C\, H^{\frac{3}{2}}} = \frac{1.5\, d\, H}{H}$$

This formula gives the exact error if $d\, H$ is an infinitesimal, and a nearly correct value of the error if $d\, H$ is some very small quantity. Assuming $d\, H = 0.005$ ft. the writer has calculated the error for different heads upon the weir. The results are shown in Fig. 28, and are expressed as percentages of the true volume flowing. The heads are on the left and the corresponding errors for the various heads at the bottom of the diagram. It will be observed that if the head is 0.1 ft., the error is $7\frac{1}{2}\%$; if the head is 1 ft., the error is 0.75%, and if the head is 5 ft., the error is only 0.15 per cent. It seems to the writer, therefore, that the required degree of precision could have been attained, and with greater certainty as to the final results, if a simpler and more direct method of measuring the heads had been followed.

It is probable, also, that some of the irregularities in the resulting coefficients are due to the method of observing the head. There is no

apparent reason why the coefficients of a standard sharp-crested weir should increase both ways from a minimum as shown in Fig. 7, and it is probable that the same cause accounts for the peculiarities in the coefficients of Experiments Nos. 1, 5, 12 and 16, whatever that cause may have been. Mr. Parmley.

The writer, agreeing with Professor Williams, is unable to understand the author's use of Bazin's formula on page 292.* After having correctly found the volume by the formula of Bazin, he apparently makes a further correction for velocity of approach, on pages 293 and 294. Even if such correction were necessary, the reason is not clear why the correction taken was $\frac{v^2}{2g}$, instead of $1.33\,\frac{v^2}{2g}$, given as the average value by Fteley and Stearns. If the writer understands Bazin's formula, no correction for velocity of approach is necessary, since that is taken account of already in the formula and the coefficients given in his table.

This suggests what seems to be an obscure point in the discussion of weirs in most text books on hydraulics. In order to use correctly any weir formula it is absolutely necessary to use it with the constants obtained by direct experiment for that formula; and it is also necessary that all the attending conditions should be the same as those under which the constants were determined. One, therefore, cannot correctly apply the Francis constants when reducing for velocity of approach by the method adopted by Fteley and Stearns, nor *vice versa*. In a similar manner, the Bazin constants are not directly applicable to the formulas obtained by Francis, and by Fteley and Stearns. If our text books had been more clear, there would have been less confusion regarding this point.

If the volume over a standard 16-ft. weir was not correctly obtained as above indicated, the subsequent work based upon these results would need correcting. It is to be hoped that the results as finally determined will be affected by no error which it is possible to eliminate.

Since, as a basis of the experiments performed at Cornell, the whole subject of weir flow is under discussion, it will probably be instructive to give a summary of the results of some of the deductions which the writer has made of the work of Francis, Fteley and Stearns, and Bazin.

As already stated, in order to obtain the most accurate results, it is necessary to duplicate the conditions of some weir for which coefficients have been determined by volumetric measurement, and then to use the same formula and the same coefficients derived from that weir. As a matter of fact, this is seldom possible in practice. The engineer is thus forced to resort to some modified method which, in his judgment, will give results most nearly correct under the circumstances.

* Since this discussion was received, the statement in the original paper has been changed, and now reads:

n = a coefficient which depends on p and h. (p = height of crest of weir above bottom of channel of approach, and h = observed head on crest.)

Mr. Parmley. The writer has constructed recently about forty weirs of various sorts, and not one of them conforms to any weir for which coefficients have been determined. Most of these weirs are in sewer outlets where the channel of approach is circular or nearly so. In all these weirs, with considerable head, the velocity of approach exceeds that of any upon which experiments have been conducted. While recognizing the desirability of constructing weirs without end contractions, in most cases it has been practically impossible, and where it could have been done, the cost precluded a weir of that form.

A close study, therefore, has been made of the work of these experimenters, in order, if possible, to discover a method which would utilize the results of all, when working with a weir unlike any which had been calibrated. One of the first things noticed was the fact that in correcting for velocity of approach, Bazin followed practically the method of Francis, but with a coefficient which gives a larger correction. There are two methods of making this correction: The theoretical method of Francis, in which the effective head, H', corrected for initial velocity of the water is

$$H' = \left[\left(H + \frac{v^2}{2g}\right)^{\frac{3}{2}} - \left(\frac{v^2}{2g}\right)^{\frac{3}{2}}\right]^{\frac{2}{3}}$$

and the empirical method used by Fteley and Stearns, in which the effective head is

$$H' = H + c\frac{v^2}{2g}$$

in which c is some numerical constant to be determined by experiment.

Now, the correction for velocity of approach, to be applied to the coefficient for flow over a weir where there is no velocity of approach, adopted by Bazin, is

$$K = 1 + 0.55\left(\frac{H}{H+P}\right)^2$$

which is identical in form with a formula, expressing the same thing, derived by Hunking and Hart:*

$$K = 1 + 0.2489\left(\frac{H}{H+P}\right)^2$$

The value of K, by Bazin, differs only by the numerical coefficient, and gives a correction 2.2 times that of the formula of Hunking and Hart; but the values given by the latter equation were found to agree within one-two hundredth of 1% of the results obtained by the use of the Francis formula for effective head, given above. It thus appears that the correction for velocity of approach, as deduced by Bazin for use in the formulas for the weirs which he experimented with, is 2.2 times the correction that Francis found necessary to the formula in the reduction of his experiments.

Journal of the Franklin Institute, Vol. 118 (Aug., 1884), p. 121.

Mr. Parmley.

The ratio $\frac{H}{H+P}$ was derived from rectangular channels and for suppressed weirs, and is, therefore, the ratio of the sectional area of the water upon the weir to the sectional area of the water in the channel of approach. Adopt the notation $\frac{a}{A}$ to represent this ratio and we may then apply the results of experiments upon suppressed weirs in rectangular channels, to weirs with end contractions in channels having any shape, provided the proper correction for end contraction be made. The formula of Francis, then, for suppressed weirs of such height that there is velocity of approach, becomes

$$Q = 3.33 \left[1 + 0.2489 \left(\frac{a}{A}\right)^2\right] L H^{\frac{3}{2}}$$

and Bazin's formula for the same case becomes

$$Q = C' \left[1 + 0.55 \left(\frac{a}{A}\right)^2\right] L H^{\frac{3}{2}}$$

in which C' = the coefficient where there is no velocity of approach.

In Bazin's formula, as thus modified, C' is not constant, as found to be nearly the case with the limited number of weirs experimented on by Francis, but varies from 3.59 for a head of 0.16 ft. to 3.28 for a head of 1.969 ft., and in the coefficient K, where the corrective term is 2.2 times that of Francis, as before stated.

Outside of the few experiments of Fteley and Stearns, the work of Francis is practically all that we have bearing on the subject of the effect of end contraction. As it was found by Fteley and Stearns to vary from 0.06 to 0.12 of the head, and as their experiments are not extensive enough for us to determine in most cases just what correction to make, the average correction of Francis seems to be the most practicable value to use, viz., 0.10 of the head deducted from the length of the weir for each end contraction. A general formula for sharp-crested weirs, either suppressed or contracted, may then be written

$$Q = C' K [L - 0.1 n H] H^{\frac{3}{2}}$$

in which n is the number of end contractions and K is the coefficient obtained by Bazin, since his experiments cover so large a number of conditions.

The values of C', as deduced by Bazin, do not fit exactly the work of American experimenters, and an investigation of this point has been made. The values of C', as deduced from the experiments of Francis, and Fteley and Stearns, after the correction for end contraction and velocity of approach had been made, are plotted in Fig. 29. The values of Bazin, for the case of no end contraction and no velocity of approach, are also plotted. In this manner the variations in the results of the experiments are set clearly before the eye. By the curve it is seen that the values of Bazin agree well with each other,

Mr. Parmley. but that he uniformly gives higher values than do either of the other experimenters. In order to write a formula capable of covering, with a fair degree of accuracy, a wide range of conditions and still give proper weight to the results of all three authorities, values of C' are taken from the curve which is drawn in an intermediate position.

Values of K for variations in the ratio $\frac{a}{A}$ from 0.01 to 0.60 are shown in Table No. 11, together with the values of C' taken from the intermediate curve of Fig. 29.

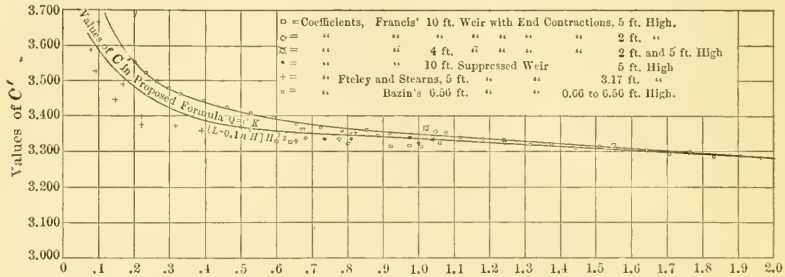


FIG. 29.

The formula, then, with the aid of the auxiliary table for C' and K, is capable of application to sharp-crested weirs, with or without end contractions, with any form of channel of approach and with a great range in the velocity of approach, these corrections being applied separately, and provided for by the coefficients in the general formula.

In order to make a direct comparison of the two methods of reducing for velocity of approach, we may consider any given quantity of flow to be calculated by the two formulas. By equating these expressions for volume we can obtain values of the coefficients that will satisfy the equation. Then let

$$Q = C'\,K\,L\,H^{\frac{3}{2}} = C'\,L\,(H + ch)^{\frac{3}{2}}$$

from which

$$K^{\frac{2}{3}}\,H = (H + ch)$$

or,

$$c = \frac{H}{h}\,(K^{\frac{2}{3}} - 1).$$

But

$$h = 0.01555\,v^2 = 0.01555\,\frac{Q^2}{A^2} = \frac{0.01555\,(C')^2\,K^2\,L^2\,H^3}{A^2}$$

$$= 0.01555\,(C')^2\,K^2\,H\,\frac{a^2}{A^2}$$

From the equation for K, given above, we have

$$A^2 = \frac{0.55\,a^2}{K - 1}$$

Substituting this value of A^2 in the equation for h, and then substituting the value of h in the equation for c and reducing gives Mr. Parmley.

$$c = \frac{35.4\ (K^{\frac{2}{3}} - 1)}{(C')^2\ K^2\ (K - 1)}$$

Since, in the second member of the last equation, there are two variables, C' and K, which are given for any depth of weir and any head upon the weir, the equation enables us to determine, after having found the volume by the equation $Q = C'\ K\ L\ H^{\frac{3}{2}}$, precisely what value of c would have been necessary to use in the formula $Q = C'\ L\ (H + ch)^{\frac{3}{2}}$ in order to obtain the same volume.

TABLE No. 11.—Values of the Coefficients in the Formula

$$Q = C'\ K\ (L - 0.1\ n\ H) H^{\frac{3}{2}}$$

a = $(L - 0.1\ n\ H)\ H$;
A = Area of water section in channel of approach;
$\frac{a}{A}$ = Ratio of area of weir section to area of section of channel of approach;
K = Constant for given value of $\frac{a}{A}$;
C' = Constant for given value of H.

$\frac{a}{A}$	K.	$\frac{a}{A}$	K.	H.	C'.
0.01	1.0001	0.31	1.0529	0.10	3.580
0.02	1.0002	0.32	1.0563	0.15	3.520
0.03	1.0005	0.33	1.0599	0.20	3.478
0.04	1.0009	0.34	1.0636	0.25	3.444
0.05	1.0014	0.35	1.0674	0.30	2.420
0.06	1.0020	0.36	1.0713	0.35	3.400
0.07	1.0027	0.37	1.0753	0.40	3.385
0.08	1.0035	0.38	1.0794	0.45	3.376
0.09	1.0044	0.39	1.0837	0.50	3.368
0.10	1.0055	0.40	1.0880	0.55	3.362
0.11	1.0066	0.41	1.0925	0.60	3.358
0.12	1.0079	0.42	1.0970	0.65	3.354
0.13	1.0093	0.43	1.1017	0.70	3.351
0.14	1.0108	0.44	1.1065	0.75	3.349
0.15	1.0124	0.45	1.1114	0.80	3.346
0.16	1.0141	0.46	1.1164	0.85	3.343
0.17	1.0159	0.47	1.1215	0.90	3.340
0.18	1.0178	0.48	1.1267	0.95	3.337
0.19	1.0198	0.49	1.1321	1.00	3.334
0.20	1.0220	0.50	1.1375	1.10	3.329
0.21	1.0243	0.51	1.1431	1.20	3.324
0.22	1.0266	0.52	1.1487	1.30	3.319
0.23	1.0291	0.53	1.1545	1.40	3.313
0.24	1.0317	0.54	1.1604	1.50	3.307
0.25	1.0344	0.55	1.1664	1.60	3.301
0.26	1.0372	0.56	1.1725	1.70	3.296
0.27	1.0401	0.57	1.1787	1.80	3.290
0.28	1.0431	0.58	1.1850	1.90	3.285
0.29	1.0463	0.59	1.1915	2.00	3.280
0.30	1.0495	0.60	1.1980	....	.....

Mr. Parmley. Fig. 30 shows the value of c for several weirs and heads as given by Fteley and Stearns* and the values for the same volumes flowing when calculated by the formula $Q = C' K L H^{\frac{3}{2}}$. The values by Fteley and Stearns are shown by broken lines, and those by the latter formula by solid lines.

It will be seen that the values of c by the two methods for a weir 1 ft. high are almost identical, but that for higher weirs, with consequently lower velocities of approach, the values by Fteley and Stearns are lower. On the other hand, by the Bazin formula with modified coefficients, the values of c increase as the weirs become higher and the velocities of approach become less. It would seem that the latter condition is

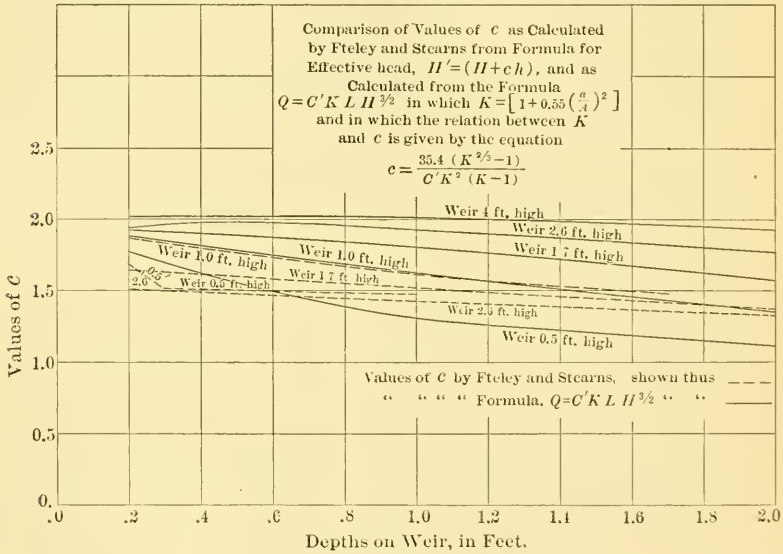


FIG. 30.

the more reasonable since there appears to be more pressure head proportionately with low velocities immediately behind the weir than with greater initial velocities. It also seems to accord with other observations of Fteley and Stearns which showed that the value of c is greater for weirs with end contraction than for suppressed weirs.

Whether or not this be true, the values by the latter formula show a regular law of variation, since c increases in every instance with an increase in the height of the weir, while the values as given by Fteley and Stearns for a weir 1.7 ft. high are intermediate between those for weirs 1 ft. and 0.5 ft. high; and for a weir 2.6 ft. high they are less than those for any of the others. Such results are anomalous, and

* *Transactions*, Am. Soc. C. E., Vol. xii, Table No. III. p. 22.

Mr. Parmley.

seem to show that the values derived by their experiments were affected by some peculiar condition which may not occur in other cases. In Table No. 12 a direct comparison of the values of c is shown for the cases having one or more end contractions, as given by Fteley and Stearns.* The values of c are calculated by the formula for c, given above. From this, together with the data in the table of Fteley and Stearns, a comparison of the velocity heads is made, with the difference between them shown in the last column.

TABLE No. 12.—Comparative Values of c in the Formula for Effect of Velocity of Approach $H' = (H + c\,h)$ as Given by Fteley and Stearns, and as Given by the Formula $c = \dfrac{35.4\,(K^{\frac{3}{2}} - 1)}{(C')^2\,K^2\,(K - 1)}$.

No. of experiment.	Height of weir.	Length of weir.	Number of end contractions.	Measured head on weir.	Value of c given by Fteley and Stearns.	Value of c given by formula.	Value of ch given by Fteley and Stearns.	Value of h by Fteley and Stearns.	Value of h by formula for c.	Column (10) deducted from column (9).
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
1	3.56	3.0	2	0.8702	†2.14	2.08	0.0042	0.0020	0.0020	0.0000
2	1.70	3.0	2	0.8612	2.26	2.03	0.0132	0.0058	0.0065	− 0.0007
3	1.00	3.0	2	0.8490	2.27	1.95	0.0254	0.0112	0.0130	− 0.0018
4	0.50	3.0	2	0.8310	2.00	1.83	0.0434	0.0217	0.0237	− 0.0020
5	3.56	3.3	1	0.5765	†2.02	2.08	0.0017	0.0008	0.0008	0.0000
6	1.00	3.3	1	0.5647	2.35	1.96	0.0135	0.0057	0.0069	− 0.0012
7	0.50	3.3	1	0.5574	1.65	1.83	0.0208	0.0126	0.0114	+ 0.0012
8	3.56	3.3	1	0.8062	†2.00	2.03	0.0040	0.0020	0.0020	0.0000
9	2.60	3.3	1	0.8036	1.99	2.05	0.0066	0.0033	0.0032	+ 0.0001
10	1.70	3.3	1	0.7969	2.16	2.01	0.0133	0.0062	0.0066	− 0.0004
11	1.00	3.3	1	0.7850	2.09	1.92	0.0252	0.0121	0.0131	− 0.0010
12	0.50	3.3	1	0.7677	1.78	1.77	0.0425	0.0239	0.0240	− 0.0001
13	3.56	3.3	1	0.9307	†1.99	2.09	0.0058	0.0029	0.0028	+ 0.0001
14	1.00	3.3	1	0.9024	2.10	1.90	0.0341	0.0162	0.0179	− 0.0017
15	0.50	3.3	1	0.8775	1.91	1.75	0.0590	0.0309	0.0337	− 0.0028
16	3.56	4.0	1	0.7048	†1.81	2.05	0.0038	0.0021	0.0019	+ 0.0002
17	0.50	4.0	1	0.6639	1.58	1.67	0.0447	0.0283	0.0268	+ 0.0015

The calculations are based on the weirs shown in *Transactions*, Am. Soc. C. E., Vol. xii, Table No. III, pp. 22 and 23.

By the last column of differences it will be observed that the values as calculated are within the limits of error in making the observation of the head upon the weir, and this fact, together with the probability that some small errors in readings were made by Fteley and Stearns, seems to make the calculated value of c practically exact in any given case.

To test the preceding work still further, and if possible remove any doubt there may be in its reliability, the volumes of discharge

* *Transactions*, Am. Soc. C. E., Vol. xii, Table No. III. pp. 22 and 23.

† Value used to determine effective depth on weir, and not the result of experiment.

Mr. Parmley. have been calculated for the various experiments made by Francis, and Fteley and Stearns, so that these calculated volumes could be compared with the volumes obtained by them from direct volumetric measurement. These results, together with the percentage of variation of calculated from measured volumes, are shown in Tables Nos. 13 to 17.

TABLE No. 13.—Experiments by Francis.

Case I.—Length of weir, 9.997 ft.; width of channel above weir, 13.96 ft.; mean depth of channel below crest of weir, 6 ft., above weir, 5.048 ft.

No. of Francis' Experiments (Mean of Nos.).	Measured head.	Actual Q, in cubic feet.	Q' by formula, in cubic feet.	$\frac{Q'}{Q}$
72, 73, 74	0.5983	15.2466	15.397	1.010
76, 77, 78	0.63823	16.7777	16.942	1.010
75	0.65525	17.4305	17.612	1.010
56, 57, 58, 59, 60, 61	0.79899	23.4304	23.66	1.009
22, 23	0.9218	28.9334	29.20	1.009
11, 17, 18, 19, 24, 31, 32, 33	0.9701	31.2479	31.466	1.007
12, 20, 21, 25, 26, 27	1.0050	32.9542	33.16	1.006
13, 14, 15, 16, 28, 29	1.0390	34.5975	34.823	1.007
30	1.0692	36.1752	36.313	1.004
5, 6, 7, 8, 9, 10	1.2476	45.5654	45.90	1.001
1, 2, 3, 4	1.5508	62.602	62.60	1.000

Case II.—Same conditions as in Case I, except that weir was divided into two bays by a central partition 2 ft. wide. Each resulting weir, 3.999 ft. long.

34, 35	1.0182	13.138	13.06	0.994

Case III.—Weir 9.997 ft. long; complete end contraction; depth of channel below crest of weir, 2.014 ft ; width of channel, 13.96 ft.

79, 80, 81, 82, 83, 84	0.6493	17.3400	17.60	1.015
62, 63, 64	0.7876	23.2658	23.518	1.011
65, 66	0.8841	27.7038	27.704	1.010
36, 37, 38, 39, 40, 41	1.0420	35.5792	35.820	1.007
42, 43	1.0753	37.2693	37.55	1.007

Case IV.—Same conditions as in Case III, except that the weir was divided into two bays by a central partition 2 ft. wide, thus making two weirs each 3.999 ft. long.

85, 86, 87, 88	0.6800	7.2739	7.352	1.011

Case V.—Suppressed weir, 9.995 ft. long; width of channel, 9.992 ft.; depth below crest of weir, 5.048 ft.

67	0.7362	21.1532	21.3?	1.009
68, 69, 70, 71	0.8099	24.4498	24.629	1.007
44, 45, 46, 47, 48, 49, 50	0.9790	32.5616	32.751	1.006
51, 52, 53, 54, 55	1.00026	33.4946	33.50	1.010

TABLE No. 14.—Experiments by Fteley and Stearns. Mr. Parmley.

Sharp-crested weir with end contractions suppressed; weir 3.17 ft. high above bottom of channel; rectangular channel of approach.

No. of Experiment.	Length of weir.	Measured head.	Actual Q, in cubic feet.	Q' by formula, in cubic feet.	$\frac{Q'}{Q}$
1	5.000	0.8198	12.750	12.66	0.9944
3	5.000	0.8118	12.466	12.519	1.004
4	5.000	0.6761	9.430	9.473	1.005
5	4.997	0.6713	9.322	9.379	1.006
6	4.996	0.5203	9.342	6.385	1.007
8	4.999	0.4761	5.547	5.590	1.008
9	4.999	0.4569	5.199	5.257	1.011
10	4.999	0.3890	4.094	4.138	1.011
12	4.994	0.3407	3.3540	3.402	1.014
13	4.998	0.3114	2.9855	2.980	1.015
14	4.999	0.2598	2.2415	2.284	1.019
15	4.999	0.2467	2.0780	2.115	1.018
17	5.000	0.2190	1.7474	1.779	1.018
18	5.000	0.2182	1.7211	1.769	1.028
19	5.000	0.2176	1.7837	1.762	0.988
20	4.998	0.1650	1.1705	1.177	1.005
21	4.995	0.1627	1.1367	1.153	1.014
22	4.995	0.1444	0.9469	0.9681	1.022
23	4.998	0.1235	0.7495	0.7452	0.994
24	4.998	0.1225	0.7526	0.7623	1.013
27	4.996	0.1009	0.5846	0.5731	0.978
28	4.996	0.1008	0.5877		
29	4.996	0.0991	0.5498	0.5582	1.015
31	4.996	0.0746	0.3652	0.3648	0.999

TABLE No. 15.

Sharp-crested weir with end contractions suppressed; weir 18.996 ft. long and 6.55 ft. high above bottom of channel.

Number of Experiment.	Measured head.	Actual Q, in cubic feet.	Q' by formula, in cubic feet.	$\frac{Q'}{Q}$
1	1.6038	130.117	130.341	1.002
2	1.4546	112.066	112.313	1.002
3	1.2981	94.192	94.652	1.005
5	1.1456	77.783	78.415	1.008
6	0.9873	62.061	62.713	1.011
7	0.9864	62.023	62.609	1.010
8	0.8191	46.760	47.491	1.016
9	0.6460	32.685	33.226	1.017
10	0.4685	20.178	20.596	1.021

TABLE No. 16.

Experiments on weirs in channel 5 ft. wide, and with height of weirs 3.56 ft. above bottom of channel; dimensions of weirs as shown in the table:

Number designating form of weir.	Number of end contractions.	Length of weir.	Distance from side of channel to the end of weir.	
			North side.	South side.
1	0	5.0	0.0	0.0
2	1	4.0	1.0	0.0
3	1	4.0	0.0	1.0
4	1	3.3	0.0	1.7
5	2	3.0	1.0	1.0
6	2	2.3	1.0	1.7

Mr. Parmley.

TABLE No. 17.

Number of Experiment.	Number designating form of weir by Table No. 16.	Measured head, in feet.	Actual Q, in cubic feet.	Q' by Formula, in cubic feet.	$\frac{Q'}{Q}$
1	1	0.1509	1.007	1.034	1.026
4	5	0.2155	1.007	1.028	1.021
6	2	0.2303	1.868	1.917	1.026
7	2	0.2691	1.868	1.911	1.023
8	3	0.2693	1.869	1.913	1.024
9	5	0.3301	1.869	1.905	1.019
10	1	0.2304	1.869	1.918	1.026
11	1	0.3368	3.283	3.343	1.018
12	2	0.3942	3.283	3.336	1.016
13	3	0.3944	3.283	3.340	1.017
14	5	0.4843	3.284	3.317	1.010
15	4	0.4498	3.284	3.337	1.016
16	6	0.5824	3.284	3.287	1.001
17	1	0.3369	3.285	3.380	1.019
18	1	0.4244	4.636	4.707	1.015
19	3	0.4978	4.636	4.705	1.015
20	4	0.5678	4.637	4.697	1.013
21	1	0.4245	4.637	4.719	1.015
22	1	0.4308	4.741	4.813	1.015
23	5	0.6215	4.736	4.761	1.005
24	6	0.7478	4.736	4.701	0.992
25	4	0.5764	4.736	4.805	1.015
26	4	0.5766	4.736	4.807	1.015
27	1	0.4302	4.731	4.814	1.015
28	1	0.5115	6.132	6.217	1.014
29	2	0.5997	6.134	6.197	1.010
30	3	0.5996	6.134	6.196	1.010
31	5	0.7398	6.134	6.130	1.000
32	4	0.6860	6.134	6.212	1.013
33	6	0.8905	6.134	6.019	0.981
34	1	0.5117	6.136	6.221	1.014
35	6	0.9548	6.796	6.640	0.977
36	1	0.5477	6.796	6.907	1.016
37	1	0.6010	7.814	7.918	1.013
38	2	0.7055	7.815	7.892	1.010
39	3	0.7064	7.815	7.905	1.011
40	5	0.8714	7.815	7.769	0.994
41	1	0.6011	7.816	7.920	1.013
42	4	0.8062	7.815	7.889	1.009
43	4	0.8063	7.804	7.886	1.010
44	1	0.6002	7.798	7.902	1.013
45	5	0.8702	7.800	7.752	0.994
Mean of 46, 47	1	0.09245	9.677	9.805	1.013
48	4	0.9317	9.667	9.755	1.009
49	4	0.9297	9.630	9.724	1.010
50	1	0.6897	9.618	9.743	1.013
51	2	0.9448	12.147	12.171	1.002
52	3	0.9432	12.147	12.137	1.001
53	1	0.8047	12.147	12.311	1.013
54	2	0.9460	12.147	12.205	1.004

It is thus seen that the formula, when applied to the experiments of Francis, and of Fteley and Stearns, agrees within about 1%, usually, even through a very wide range of conditions. For the most part, the formula gives results too large for these experiments. If applied to the work of Bazin, on the other hand, the results for small heads will be found to be too small by about the same percentage, and for larger heads the error approaches zero. The formula, with the coefficients as they stand, gives about equal weight to the work of all these

experimenters, and may therefore be used with a good deal of confidence in cases where the conditions are not exactly like those of weirs actually tested. In general, it will be noticed that the percentage of variation is greater for small depths than for large depths on the weir. As, for small heads, the experimental errors are also proportionately greater, the errors of the formula are of less consequence than they would be if they increased with the depth on the weir. Mr. Farmley.

A serious difficulty with the method of calculating the effective head by the formula $H' = H + c\,h$ is the fact that the value of c varies, not only with the varying head, but with every depth of weir, and, as Fteley and Stearns found, with the condition as to end contraction. This difficulty is in part obviated by the foregoing process.

To the writer's mind, the conclusion to be drawn from the foregoing discussion is that the methods adopted by Francis and by Bazin when investigating the effect of velocity of approach are preferable to the method used by Fteley and Stearns. This conclusion seems to be confirmed by the fact that it is possible to write a single formula, with a supplemental table of values of C', depending on the head upon the weir, and a table of values of K, depending on the relative areas of water section on the weir and in the channel of approach, which will give results nearly approximating the true quantities of discharge for a very wide range of conditions.

It is to be hoped that the experiments, so well begun at Cornell University, may be continued until the subject of weir flow is taken out of the cloud of uncertainty in which at present it is enveloped.

George T. Nelles, M. Am. Soc. C. E. (by letter).—Only those engineers, whose practice has shown them the difficulties and uncertainties involved in the determination of the flow of water over dams of practical form and dimensions, will ever fully realize the great service rendered to the profession by the experiments of Bazin and those made for the U. S. Board of Engineers on Deep Waterways by the author, at the Cornell Hydraulic Laboratory. Mr. Nelles.

The Cornell experiments are presented in a practical and useful form and admit of an easy and direct application to the solution of questions concerning the flow of water over dams, met with in every-day practice by the working engineer.

Early in the course of a recent investigation to determine the probable effect of the construction of several large dams in the Tennessee River, for the purpose of producing slack-water navigation, the writer recognized the inapplicability of the existing published data, based on small-scale experiments, and made a systematic effort to secure information of a more practical nature, founded on actual observations on the flow over existing dams. His efforts in this direction, which comprised a study of the available literature on the subject, and correspondence with many of the most eminent hydraulic

Mr. Nelles. engineers of this country and Europe, met with small success, and exhibited the paucity of exact knowledge of this nature.

The best results obtained, relating to the free discharge into air, were as follows:

(1) From Dwight Porter, M. Am. Soc. C. E., five observations, made in 1894 by two students of the Massachusetts Institute of Technology, on the flow over a dam in the Blackstone River, at Albion, Mass. The dam is 217 ft. long, with a longitudinal timber crest 12 ins. wide; the lower face is vertical, and the upper face is covered with rip-rap stone, sloping at about 30° with the horizontal; the height of the dam is about 8 ft. The discharge was measured by meter at a section 500 ft. below the dam. The head was measured by a hook gauge, located 20 ft. above the crest of the dam.

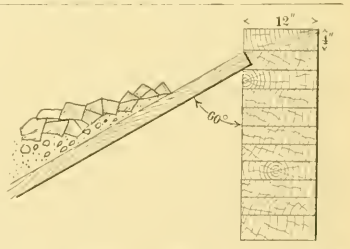
These observations when applied to the Bazin formula

$$Q = m\ L\ h\ \sqrt{2\,g\,h} \qquad (a),$$

in which m represents the coefficient of discharge, L the crest length of the dam and h the observed head, give the values for the coefficient m shown in Table No. 18.

TABLE No. 18

Observation.	Observed head, in feet.	Coefficient m.
(1)	0.735	0.439
(2)	0.867	0.441
(3)	0.829	0.456
(4)	0.973	0.491
(5)	1.025	0.470



Concerning these results, Professor Porter says: "The coefficients are sufficiently high and discordant to be unsafe for acceptance unless otherwise confirmed."

(2) From W. H. Bixby, M. Am. Soc. C. E., Major, Corps of Engineers, U. S. A., four observations on the flow over dams in the Muskingum River, Ohio. The discharge was obtained by carefully sounding the area of a section 500 ft. above each dam, and measuring the velocity by long rod floats. The mean velocity was deduced from the observed, by the formula given in Johnson's "Surveying."

The dams are constructed of timber cribs, filled with stone, and have planked sloping crests. The upper face is 10 ft. long, with a slope of 1 ft. The lower face is 30 ft. long, with a slope of 4 ft.*

* *The Engineering Record*, March 8th, 1890, page 216.

Mr. Nelles.

Table No 19 gives the results of these observations.

TABLE No. 19.

Number of dam.	Length on crest, in feet.	Mean height, in feet.	Area of discharge section, in square feet.	Discharge, in cubic feet per second.	Mean velocity, in feet per second.	Fall over dam, in feet.	Observed depth on crest, in feet.	Coefficient, in the Bazin formula (m).
3	848	12.6	7 765	18 118	2.333	8.00	2.86	0.551
4	535	15.9	8 360	25 559	3.045	6.70	4.66	0.589
7	472	14.2	8 230	21 015	2.553	7.00	4.40	0.600
8	515	16.0	7 330	22 310	3.044	5.16	5.90	0.376

T. C. Clarke, M. Am. Soc. C. E., gives certain results* concerning the flow over a dam in the Ottawa River, from which the following is abstracted. The dam is 30 ft. high, built of stone-filled timber cribs, with upper and lower faces planked and sloping at 3 horizontal to 1 vertical. The data are shown in Table No. 20.

TABLE No. 20.

Length, in feet.	Depth on crest, in feet.	Discharge, in cubic feet per second.	Coefficient in the Bazin formula (m).
1 600	2.5	26 000	0.512
1 760	10.0	190 000	0.425

In a paper read before the American Society of Mechanical Engineers in 1897, Professor R. C. Carpenter, of Cornell University, gave the results of a number of observations made by him at the Bridgeport Pumping Station, Chicago, on the flow over a thin vertical weir, 30 ft. long and 8.7 ft. high, with a sharp crest.

The discharge was measured by carefully conducted rod float observations, and checked by the known capacity of the basin. After correcting for two end contractions, Professor Carpenter gives the figures shown in Table No. 21.

TABLE No. 21.

Observed heads, in feet, corrected for velocity of approach	0.4	0.5	0.6	0.7	0.8	0.9	1.0
Coefficient C in Francis' formula	3.657	3.609	3.577	3.560	3.545	3.537	3.529

The coefficients given by the Massachusetts Institute of Technology observations and the Muskingum River observations have been plotted on the diagram, Fig. 31, in comparison with Bazin's Series No. 178; and appear to justify the early conclusions drawn from the Bazin experiments, that for weirs with sloping faces, the coefficients increase

* *Transactions*, Am. Soc. C. E., Vol. xxxiv, page 508.

Mr. Nelles. with the head. When, however, we compare these coefficients with the Cornell experiments, of which Series No. 7 is taken as a type, and shown in Fig. 31, we are met by complications which give rise to just doubts as to the accuracy of the observations in question, or the applicability of experiments, even on the large scale of those carried out at Cornell, to the flow over structures differing in shape and dimensions from the experimental forms. While both causes may combine to produce the apparent discrepancies, the former is doubtless the leading factor. The condition of the nappe, if known, would probably account, in some degree, for the discrepancies noted.

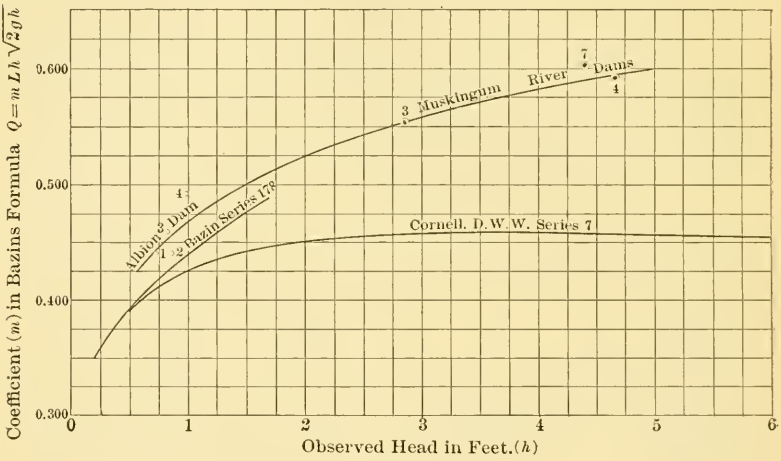


Fig. 31.

The heads corresponding to the curve representing the Cornell D. W. W. Series No. 7 have been corrected for velocity of approach.

Flow over Submerged Dams.

The practical solution of most questions concerning the free flow over weirs and dams of the usual forms is rendered comparatively easy and simple by the experiments recorded in this paper. The solution of questions concerning the flow over submerged or drowned weirs is still, however, involved in doubt and uncertainty, although the recent experiments by Bazin, covering this form of flow, have added greatly to our knowledge, and afford a basis for a comprehensive study, which will doubtless eventually lead to a better, if not a complete, understanding of the governing principles. A knowledge of this form of flow, which is of relatively less importance to engineers in general than a knowledge of free flow, is nevertheless essential in the study of various engineering problems.

Mr. Nelles.

The experiments of Francis, Fteley and Stearns, Bazin, and others, have determined, within the limits of their experiments, the law of variation of the coefficient of discharge for submerged flow over thin sharp-crested vertical weirs. The results of and the deductions from the experiments of Francis, and Fteley and Stearns, are to be found in the *Transactions* of this Society,* to which reference is made. Bazin has applied the results of his experiments on submerged flow over weirs with sharp crests, in thin partition, to the same formula he used for free discharge into air,

$$Q = m\,L\,h\,\sqrt{2\,g\,h} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (a)$$

and has deduced the two expressions (Equations 11 and 12, page 265), for the coefficient of discharge m. In these equations (11 and 12) m' represents the coefficient of free discharge for a similar weir of the same height, with the same depth h of water in the pool above the weir on the crest. The depth of the water in the pool below the weir on the crest is represented by h_1; p represents the height of the weir, and $z = h - h_1 =$ the difference in level of the water surface above and below the weir. These expressions, in connection with the well-determined coefficients of free discharge given in Table No. 2 afford an easy and safe means of obtaining the coefficient for submerged flow for the conditions and limits comprised in Bazin's experiments, Series Nos. 64 to 85, 4th article, 1894.

In his 5th article, 1896, Bazin treats of the submerged flow over vertical weirs with flat crests (*déversoirs à poutrelles*), made up of square timbers of the same width as the crest. Concerning these experiments he says:

"Independent of the experiments we have just studied (experiments on the free flow over similar weirs), there have been made, by raising the level of the water surface below the weirs, four other series, on weirs 0.75 m. (2.46 ft.) and 0.35 m. (1.15 ft.) high above the bed of the channel, with crests 0.20 m. (0.66 ft.) and 0.10 m. (0.33 ft.) wide.

"They are entirely analogous to those which were made on vertical weirs in thin partition† (Series 64 to 85), and exactly the same methods have been followed in measuring the discharge, pressure, etc.

"The discharge was regulated to correspond to constant heads of 0.10 m , 0.15 m. * * * * 0.40 m., 0.45 m (0.325 ft , 0.492 ft. * * * * 1.148 ft., 1.312 ft.), on the standard weir of comparison. When the discharge corresponding to a given head became constant, then the level of the water surface below the weir was successively changed to $0.04p$, $0.08p$, $0.12p$, etc. (p = height of weir), both above and below the crest, as was done in the similar experiments on weirs in thin partition. Finally, a similar series was executed on a vertical weir 2.0 m. (6.56 ft.) wide."

* *Transactions*, Am. Soc. C. E., Volumes xii, xiii and xiv.

† In his articles Bazin refers to weirs similar in form to his standard weir, as weirs in thin partition (*Déversoirs en mince paroi*). These weirs are made up of horizontal timbers about 4 ins. square, the crest being formed by an iron plate ¼ in. thick, in the prolongation of the back of the weir and extending 4 ins. above the timbers.

Weirs of other form, with sharp crest, Series 195, 196 and 197, he designates as weirs with sharp crest (*Déversoirs à vive arête*).

Mr. Nelles.

BAZIN'S EXPERIMENTS.—SERIES NOS. 120 TO 123.

Nappe wetted underneath, with water retained in pool below weir. Description of weirs under observation: Height, 0.75 m. = 2.46 ft. and 0.35 m. = 1.15 ft. Crests, 0.20 m. = 0.66 ft. and 0.10 m. = 0.33 ft. wide. Length, 2.0 m. = 6.56 ft.

Dimensions of standard weir used for measuring the discharge: Height, 1.135 m. = 3.72 ft. Length, 2.0 m. = 6.56 ft.

(1)	(2)	(3)	(4)	(5)
	Observed Heads, in Feet.		Height of the retained water with reference to crest: + = above, − = below. (h_1)	Coefficients of Discharge.
	On weir under consideration. (h.)	On standard weir of comparison. (H.)		For the Bazin Formula a ($Q = m\,Lh\sqrt{2gh}$)
	Bazin's Series No. 120.			
1	0.388	0.329	−0.199	0.3379
2	0.394	0.335	−0.097	0.3391
3	0.395	0.336	−0.002	0.3388
4	0.392	0.332	+0.191	0.3373
5	0.402	0.331	+0.294	0.3241
6	0.447	0.331	+0.392	0.2758
7	0.539	0.333	+0.494	0.2100
1	0.548	0.496	−0.392	0.3684
2	0.550	0.496	−0.200	0.3671
3	0.549	0.489	+0.000	0.3602
4	0.558	0.498	+0.098	0.3605
5	0.552	0.484	+0.187	0.3525
6	0.569	0.498	+0.296	0.3502
7	0.569	0.486	+0.392	0.3378
8	0.605	0.498	+0.490	0.3194
9	0.680	0.492	+0.589	0.2629
10	0.760	0.498	+0.686	0.2269
1	0.699	0.657	−0.394	0.3878
2	0.701	0.657	−0.197	0.3858
3	0.710	0.660	−0.001	0.3810
4	0.717	0.662	+0.199	0.3774
5	0.723	0.664	+0.296	0.3737
6	0.723	0.664	+0.295	0.3740
7	0.722	0.655	+0.388	0.3683
8	0.742	0.663	+0.492	0.3589
9	0.765	0.654	+0.593	0.3369
10	0.837	0.660	+0.687	0.2980
11	0.909	0.658	+0.783	0.2621
1	0.842	0.826	−0.589	0.4126
2	0.841	0.818	−0.395	0.4077
3	0.846	0.823	−0.198	0.4081
4	0.852	0.825	−0.004	0.4053

(1)	(2)	(3)	(4)	(5)
	Observed Heads, in Feet.		Height of the retained water with reference to crest: + = above, − = below. (h_1)	Coefficients of Discharge.
	On weir under consideration. (h.)	On standard weir of comparison. (H.)		For the Bazin Formula ($Q = m\,Lh\sqrt{2gh}$)
5	0.854	0.824	+0.100	0.4084
6	0.861	0.829	+0.189	0.4021
7	0.862	0.828	+0.193	0.4009
8	0.867	0.830	+0.290	0.3981
9	0.861	0.819	+0.291	0.3952
10	0.861	0.812	+0.419	0.3905
11	0.890	0.820	+0.587	0.3762
12	1.006	0.829	+0.786	0.3184
13	1.131	0.819	+0.980	0.2621
1	0.979	0.998	−0.788	0.4386
2	0.974	0.983	−0.396	0.4318
3	0.987	0.995	−0.199	0.4316
4	0.984	0.985	0.000	0.4266
5	0.990	0.982	+0.197	0.4207
6	1.000	0.989	+0.294	0.4191
7	1.003	0.982	+0.392	0.4130
8	1.005	0.988	+0.393	0.4151
9	1.006	0.979	+0.491	0.4088
10	1.014	0.991	+0.492	0.4112
11	1.020	0.984	+0.588	0.4034
12	1.024	0.987	+0.595	0.4034
13	1.046	0.988	+0.658	0.3917
14	1.102	0.987	+0.788	0.3617
15	1.158	0.988	+0.887	0.3318
16	1.226	0.994	+0.980	0.3111
17	1.288	0.988	+1.082	0.2862
18	1.358	0.982	+1.182	0.2657
1	1.111	1.157	−0.584	0.4541
2	1.117	1.158	−0.397	0.4507
3	1.118	1.153	−0.193	0.4467
4	1.117	1.149	0.000	0.4473
5	1.127	1.149	+0.196	0.4414
6	1.130	1.148	+0.293	0.4378
7	1.142	1.157	+0.390	0.4346
8	1.136	1.150	+0.487	0.4294
9	1.142	1.147	+0.493	0.4304
10	1.156	1.152	+0.588	0.4251
11	1.158	1.150	+0.593	0.4227
12	1.172	1.151	+0.688	0.4158
13	1.174	1.154	+0.690	0.4158
14	1.211	1.150	+0.788	0.3955
15	1.279	1.153	+0.881	0.3653

Mr. Nelles.

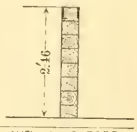
(1)	(2) h	(3) H	(4) h_1	(5) m
16	1.335	1.151	+0.984	0.3420
1	1.243	1.312	−0.394	0.4644
2	1.252	1.317	−0.198	0.4620
3	1.253	1.316	−0.002	0.4615
4	1.250	1.313	+0.195	0.4561
5	1.269	1.308	+0.597	0.4481
6	1.274	1.308	+0.490	0.4451
7	1.288	1.312	+0.590	0.4495
8	1.304	1.317	+0.687	0.4529
9	1.306	1.315	+0.689	0.4201
10	1.326	1.314	+0.784	0.4345
11	1.332	1.315	+0.785	0.4220
12	1.397	1.314	+0.889	0.3907
13	1.446	1.320	+0.985	0.3734
1	1.376	1.479	−0.390	0.4787
2	1.382	1.478	−0.200	0.4753
3	1.396	1.491	+0.001	0.4746
4	1.395	1.479	+0.197	0.4688
5	1.400	1.474	+0.396	0.4635
6	1.423	1.479	+0.589	0.4555
7	1.461	1.480	+0.780	0.4381

Bazin's Series No. 121.

1	0.386	0.328	−0.193	0.3395
2	0.387	0.329	−0.096	0.3396
3	0.388	0.329	+0.041	0.3383
4	0.388	0.330	+0.086	0.3398
5	0.389	0.331	+0.183	0.3391
6	0.395	0.329	+0.271	0.3295
7	0.423	0.328	+0.371	0.2963
8	0.496	0.325	+0.469	0.2288
9	0.581	0.333	+0.564	0.1883
10	0.667	0.331	+0.655	0.1516
1	0.540	0.495	−0.462	0.3760
2	0.542	0.492	−0.374	0.3714
3	0.546	0.493	−0.187	0.3681
4	0.547	0.492	−0.001	0.3665
5	0.550	0.493	+0.087	0.3632
6	0.550	0.495	+0.184	0.3653
7	0.555	0.494	+0.273	0.3604
8	0.562	0.494	+0.369	0.3532
9	0.579	0.494	+0.455	0.3383
10	0.624	0.493	+0.557	0.2999
11	0.709	0.494	+0.647	0.2492
12	0.780	0.494	+0.735	0.2158
	0.689	0.660	−0.175	0.3987
	0.690	0.661	−0.091	0.3993
	0.699	0.661	+0.184	0.3917
4	0.701	0.659	+0.281	0.3878
5	0.711	0.660	+0.370	0.3809
6	0.716	0.659	+0.459	0.3758
7	0.732	0.658	+0.561	0.3635
8	0.768	0.661	+0.649	0.3405
9	0.841	0.661	+0.742	0.2969
10	0.911	0.657	+0.833	0.2605
11	0.988	0.658	+0.929	0.2308
1	0.829	0.824	+0.001	0.4219
2	0.839	0.825	+0.181	0.4147
3	0.839	0.822	+0.281	0.4127
4	0.847	0.829	+0.368	0.4111
5	0.852	0.828	+0.458	0.4076
6	0.863	0.827	+0.557	0.3992
7	0.871	0.828	+0.618	0.3934
8	0.875	0.819	+0.654	0.3852

(1)	(2) h	(3) H	(4) h_1	(5) m
9	0.906	0.825	+0.732	0.3700
10	0.975	0.828	+0.832	0.3322
11	1.040	0.823	+0.923	0.2994
12	1.112	0.823	+1.020	0.2710
13	1.189	0.824	+1.112	0.2455
1	0.961	0.986	+0.178	0.4432
2	0.968	0.991	+0.281	0.4460
3	0.967	0.986	+0.375	0.4377
4	0.979	0.987	+0.463	0.4315
5	0.986	0.987	+0.558	0.4269
6	1.001	0.988	+0.652	0.4173
7	1.020	0.989	+0.744	0.4067
8	1.019	0.989	+0.745	9.4073
9	1.047	0.985	+0.829	0.3894
10	1.113	0.992	+0.923	0.3583
11	1.177	0.989	+1.017	0.3289
12	1.248	0.989	+1.113	0.3007
13	1.314	0.984	+1.205	0.2760
1	1.093	1.147	+0.281	0.4597
2	1.100	1.150	+0.368	0.4569
3	1.105	1.151	+0.464	0.4544
4	1.115	1.152	+0.557	0.4489
5	1.121	1.150	+0.651	0.4439
6	1.134	1.151	+0.736	0.4368
7	1.158	1.152	+0.828	0.4235
8	1.168	1.147	+0.876	0.4154
9	1.190	1.150	+0.925	0.4054
10	1.255	1.150	+1.018	0.3751
11	1.315	1.151	+1.113	0.3493
12	1.381	1.151	+1.204	0.3249
13	1.439	1.149	+1.293	0.3051
1	1.230	1.317	+0.462	0.4742
2	1.237	1.316	+0.557	0.4693
3	1.242	1.314	+0.653	0.4654
4	1.257	1.315	+0.743	0.4586
5	1.268	1.319	+0.834	0.4484
6	1.294	1.311	+0.923	0.4358
7	1.344	1.311	+1.020	0.4126
8	1.371	1.316	+1.063	0.4024
9	1.398	1.312	+1.115	0.3890
10	1.453	1.314	+1.204	0.3684
11	1.509	1.315	+1.294	0.3479
1	1.336	1.478	+0.659	0.4995
2	1.376	1.476	+0.747	0.4763
3	1.395	1.485	+0.829	0.4721
4	1.405	1.477	+0.922	0.4628
5	1.441	1.476	+1.024	0.4447
6	1.495	1.478	+1.114	0.4222
7	1.542	1.478	+1.203	0.4029
8	1.587	1.477	+1.292	0.3861

Bazin's Series No. 122.

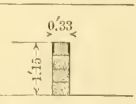


1	0.360	0.330	−0.187	0.3792
2	0.353	0.322	−0.098	0.3780
3	0.361	9.328	0.000	0.3755
4	0.371	0.330	+0.095	0.3659
5	0.372	0.327	+0.192	0.3571
6	0.376	0.337	+0.294	0.3681
1	0.495	0.502	−0.381	0.4354
2	0.495	0.489	−0.198	0.4187
3	0.504	0.494	−0.002	0.4155
4	0.515	0.495	+0.198	0.4036

Mr. Nelles.

(1)	(2) h	(3) H	(4) h_1	(5) m
5	0.523	0.493	+0.293	0.3923
6	0.579	0.492	+0.390	0.3852
7	0.756	0.494	+0 488	0.2259
1	0.631	0.658	−0.401	0.4529
2	0.634	0.658	−0.199	0.4505
3	0.640	0.657	0.000	0.4476
4	0.644	0.650	+0.197	0.4324
5	0.654	0.654	+0.292	0.4251
6	0.718	0.655	+0.396	0.3713
7	0.808	0.658	+0.596	0.3130
1	0.751	0.813	−0.590	0.1805
2	0.768	0.824	−0.393	0.4734
3	0.766	0.821	−0.199	0.4733
4	0.771	0.819	+0.002	0.4667
5	0.779	0.813	+0.196	0.4543
6	0.811	0.823	+0.299	0.4126
7	0.853	0.812	+0.387	0.3959
8	0.887	0.817	+0.489	0.3758
8	0.931	0.823	+0.592	0.3541
10	1.016	0.815	+0.784	0.3059
1	0.880	0.987	−0.791	0.5064
2	0.879	0.980	−0.589	0.5117
3	0.884	0.983	−0.411	0.5001
4	0.890	0.991	−0.398	0.5010
5	0.889	0.982	−0.199	0.4946
6	0.897	0.978	−0.001	0.4859
7	0.912	0.986	+0.095	0.4792
8	0.960	0.983	+0.196	0.4412
9	0.984	0.986	+0.291	0.4279
10	1.006	0.983	+0.391	0.4114
11	1.027	0.983	+0.490	0.3991
12	1.059	0.978	+0.587	0.3784
13	1.099	0.986	+0.688	0.3624
14	1.135	0.984	+0.784	0.3447
15	1.231	0.985	+0.977	0.3048
1	1.009	1.153	−0.585	0.5214
2	1.012	1.147	−0.408	0.5150
3	1.018	1.148	−0.201	0.5118
4	1.077	1.151	+0.001	0.4722
5	1.118	1.149	+0.192	0.4453
6	1.156	1.147	+0.388	0.4227
7	1.206	1.151	+0.585	0.3983
8	1.234	1.149	+0.688	0.3839
9	1.264	1.152	+0.783	0.3715
10	1.344	1.149	+0.980	0.3374
1	1.146	1.307	−0.401	0.5221
2	1.202	1.313	−0.201	0.4892
3	1.242	1.311	−0.006	0.4644
4	1.270	1.312	+0.189	0.4497
5	1.303	1.312	+0.387	0.4322
6	1.354	1.312	+0.595	0.4087
1	1.348	1.479	−0.395	0.4934
2	1.370	1.479	−0.192	0.4819
3	1.393	1.476	+0.004	0.4683

Series No. 122.



1	0.355	0.331	−0.282	0.3885
2	0.358	0.330	−0.188	0.3845
3	0.354	0.327	−0.092	0.3827
4	0.362	0.329	+0.003	0.3758
5	0.361	0.327	+0.094	0.3729
6	0.370	0.331	+0.180	0.3654
7	0.382	0.327	+0.279	0.3420
8	0.441	0.330	+0.365	0.2792

(1)	(2) h	(3) H	(4) h_1	(5) m
1	0.496	0.502	−0.373	0.4339
2	0.494	0.494	−0.281	0.4278
3	0.496	0.500	−0.184	0.4302
4	0.497	0.499	−0.091	0.4277
5	0.496	0.494	+0.005	0·4235
6	0.505	0.494	+0.095	0.4153
7	0.505	0.493	+0.181	0.4128
8	0.516	0.499	+0.280	0.4054
9	0.537	0.503	+0.3.0	0.3866
10	0.599	0.501	+0.466	0.3269
11	0.655	0.496	+0.561	0.2813
12	0.721	0.501	+0.649	0.2470
1	0.622	0.658	−0.181	0.4611
2	0.632	0.666	−0.091	0.4595
3	0.627	0.656	−0.002	0.4547
4	0.639	0.663	+0.090	0.4488
5	0.634	0.657	+0.179	0.4480
6	0.641	0.657	+0.269	0.4408
7	0.657	0.657	+0.368	0.4244
8	0.707	0.658	+0.467	0.3822
9	0.756	0.658	+0.557	0 3460
10	0.806	0.665	+0.650	0.3190
11	0.861	0.662	+0.742	0.2863
1	0.755	0.822	+0.001	0.4827
2	0.762	0.823	+0.097	0.4776
3	0.760	0.819	+0.179	0.4749
4	0.766	0.818	+0.272	0.4688
5	0.782	0.822	+0.368	0.4579
6	0.832	0.819	+0.469	0.4157
7	0.869	0.824	+0.559	0.3922
8	0.902	0.825	+0.649	0.3716
9	0.948	0.825	+0.742	0.3449
10	0.998	0.817	+0.843	0.3141
11	1.060	0.825	+0.928	0.2918
1	0.884	0.987	+0.182	0.5018
2	0.894	0.993	+0.277	0.4967
3	0.923	0.987	+0.369	0.4708
4	0.959	0.981	+0.462	0.4401
5	0.993	0.995	+0.560	0.4263
6	1.018	0.986	+0.648	0.4048
7	1.059	0.995	+0.743	0.3878
8	1.090	0.984	+0.838	0.3635
9	1.134	0.983	+0.922	0.3435
10	1.186	0.984	+1.016	0.3211
1	1.068	1.153	+0.365	0.4783
2	1.100	1.149	+9.466	0.4549
3	1.123	1.158	+0.557	0.4471
4	1.141	1.150	+0.651	0.4315
5	1.177	1.154	+0.743	0.4137
6	1.205	1.152	+0.839	0.3984
7	1.236	1.155	+0.923	0.3850
8	1.283	1.157	+1.018	0.3646
9	1.321	1.148	+1.112	0.3150
10	1.387	1.152	+1.208	0.3225
1	1.218	1.314	+0.559	0.4619
2	1.269	1.317	+0.653	0.4520
3	1.290	1.314	+0.746	0.4388
4	1.318	1.317	+0.835	0.4268
5	1.347	1.317	+0.923	0.4130
6	1.387	1.315	+1.025	0.3940
7	1.423	1.315	+1.110	0.3880
8	1.475	1.321	+1.203	0.3640
1	1.406	1.480	+0.655	0.4632
2	1.447	1.481	+0.745	0.4581
3	1.444	1.478	+0.831	0.4444
4	1.465	1.483	+0.926	0.4365
5	1.498	1.479	+1.018	0.4204
6	1.526	1.478	+1.110	0.4085
7	1.572	1.477	+1.202	0.3909

Bazin's Experiments.—Table showing the Heads corresponding to the same Discharge over the Four Weirs just considered, for Various Elevations of the Surface of the Water retained below the Weir.

A denotes weir 0.75 m. = 2.46 ft. high; with crest 0.20 m. = 0.66 ft. wide.
B " " 0.35 m. = 1.15 " " " " 0.20 m. = 0.66 " "
C denotes weir 0.75 m. = 2.46 ft. high; with crest 0.10 m. = 0.33 ft. wide
D " " 0.35 m. = 1.15 " " " " 0.10 m. = 0.33 " "

(1) Head on the standard weir of comparison (H) in feet.	(2) Description of weir under consideration.	(3) Head on similar weir when flow is not affected by water below.	Head, in Feet, on the Weirs under Consideration. Height of the water surface below the weirs, above the level of the crest h_1. 0	0.098	0.197	0.295	0.394	0.492	0.591	0.689	0.787	0.886	0.984	1.083	1.181	Remarks.
0.328.....	*A*	0.387	0.391	0.391	0.394	0.404	0.443	0.542	.....	.....	.....	.....	.....	.....	.....	The nappe was detached from the crest when head became about 0.4 ft.
	B	0.384	0.387	0.391	0.414	0.404	0.439	0.509	.....	.....	.....	.....	.....	.....	.....	
	C	0.358	0.361	0.364	0.371	0.384	0.456	0.456	.....	.....	.....	.....	.....	.....	.....	
	D	0.351	0.361	0.364	0.371	0.387	0.446	0.446	.....	.....	.....	.....	.....	.....	.....	
0.492.....	*A*	0.548	0.551	0.558	0.561	0.568	0.578	0.608	0.676	0.787	.....	.....	.....	.....	.....	The nappe was detached from crest when the head became about 0.53 ft.
	B	0.548	0.551	0.551	0.555	0.558	0.568	0.598	0.659	0.742	.....	.....	.....	.....	.....	
	C	0.499	0.502	0.509	0.518	0.532	0.598	0.598	0.598	0.598	.....	.....	.....	.....	.....	
	D	0.479	0.495	0.499	0.509	0.522	0.551	0.626	0.676	0.676	.....	.....	.....	.....	.....	
0.656.....	*A*	0.699	0.709	0.715	0.719	0.725	0.732	0.745	0.765	0.830	0.919	0.919	.....	.....	.....	The nappe was detached from the crest when the head became about 0.7 ft.
	B	0.689	0.689	0.692	0.699	0.706	0.709	0.719	0.745	0.794	0.866	0.948	.....	.....	.....	
	C	0.630	0.636	0.643	0.653	0.666	0.719	0.762	0.810	0.856	0.856	0.856	.....	.....	.....	
	D	0.614	0.626	0.633	0.636	0.643	0.673	0.715	0.768	0.820	0.820	0.820	.....	.....	.....	
0.820.....	*A*	0.830	0.853	0.856	0.859	0.866	0.876	0.883	0.896	0.926	0.981	1.060	1.145	1.145	.....	The nappe was detached from the crest when the head became about 0.8 ft.
	B	0.827	0.830	0.833	0.837	0.840	0.843	0.850	0.869	0.889	0.945	1.014	1.083	1.174	.....	
	C	0.752	0.775	0.784	0.794	0.817	0.850	0.893	0.929	0.977	1.027	1.027	1.027	1.027	.....	
	D	0.745	0.752	0.758	0.762	0.772	0.797	0.833	0.873	0.916	0.971	1.027	1.027	1.027	.....	
0.984.....	*A*	0.964	0.987	0.991	0.994	1.000	1.007	1.014	1.027	1.047	1.103	1.168	1.227	1.292	1.355	The nappe was detached from the crest when the head became about 0.95 ft.
	B	0.958	0.958	0.958	0.961	0.964	0.971	0.981	0.991	1.014	1.033	1.086	1.151	1.224	1.299	
	C	0.879	0.906	0.926	0.954	0.977	1.004	1.033	1.063	1.103	1.145	1.188	1.237	1.237	1.237	
	D	0.873	0.873	0.886	0.886	0.896	0.932	0.945	1.000	1.041	1.086	1.132	1.171	1.224	1.224	
1.148.....	*A*	1.099	1.119	1.126	1.129	1.132	1.142	1.145	1.161	1.178	1.214	1.276	1.342	1.414	1.414	The nappe was detached from the crest when the head became about 1.1 ft.
	B	1.096	1.096	1.096	1.096	1.096	1.099	1.109	1.116	1.129	1.145	1.174	1.227	1.289	1.361	
	C	1.017	1.083	1.093	1.116	1.136	1.151	1.178	1.207	1.234	1.267	1.309	1.355	1.355	1.355	
	D	1.080	1.080	1.080	1.080	1.080	1.073	1.099	1.126	1.155	1.184	1.221	1.257	1.302	1.361	
1.312.....	*A*	1.230	1.254	1.257	1.260	1.267	1.270	1.279	1.292	1.305	1.332	1.381	1.451	1.451	1.451	The nappe was detached from the crest when the head became about 1.2 ft.
	B	1.224	1.224	1.224	1.224	1.224	1.224	1.230	1.244	1.250	1.263	1.286	1.325	1.381	1.441	
	C	1.155	1.240	1.257	1.276	1.289	1.309	1.328	1.375	1.371	1.401	1.444	1.476	1.476	1.476	
	D	1.221	1.221	1.221	1.221	1.221	1.221	1.234	1.254	1.279	1.302	1.335	1.368	1.408	1.457	

Mr. Nelles. "In the experiments with weirs in thin partition, we have had recourse to Equations 11 and 12" (page 265), "to deduce the value of the coefficient m, but the presence of the sill introduces such complications that these equations are no longer perfectly applicable. The constants, which appear in the equations, vary so greatly with the head and width of crest, that it becomes impossible to establish a simple and practical formula.

"We must, therefore, content ourselves with a general *résumé* in a table showing the heads on the four weirs under consideration, corresponding to constant discharges and varying elevation of the water surface below the weirs. The corresponding head, where the flow is not affected by the retained water, is also shown."

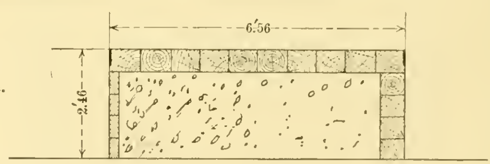
"We see that for the same discharge the heads vary notably from one weir to the other. The question is singularly complicated by the detachment of the nappe, which leaves the sill when the head attains certain limits depending on the width of the crest. The head and coefficient are then modified suddenly. This limit depends on the height of the retained water, and is not the same, even under fixed conditions. We cannot, therefore, establish as precise formulas as in the more simple case of the flow over weirs in thin partition."

Bazin's Experiments.—Series No. 124.

Nappe wetted underneath, with water retained in pool below weir. Height of weir under observation, 0.75 m. = 2.46 ft. Width of crest, 2.0 m. = 6.56 ft. Length, 2.0 m. = 6.56 ft.

Height of standard weir of comparison, 0.75 m. = 2.46 ft. Length, 2.0 m. = 6.56 ft.

Bazin's Series No. 124.



(1)	(2) h	(3) H	(4) h_1	(5) m	(1)	(2) h	(3) H	(4) h_1	(5) m
1	0.398	0.325	+0.098	0.3199	1	0.961	0.815	+0.593	0.2325
2	0.403	0.330	+0.197	0.3227	2	0.966	0.814	+0.692	0.3289
3	0.399	0.325	+0.299	0.3187	3	0.971	0.820	+0.792	0.3299
4	0.451	0.328	+0.394	0.2678	4	1.024	0.817	+0.890	0.3034
5	0.526	0.326	+0.491	0.2111	5	1.094	0.814	+0.989	0.2736
					6	1.170	0.812	+1.084	0.2462
1	0.598	0.494	+0.295	0.3228					
2	0.590	0.492	+0.393	0.3264	1	1.158	0.990	+0.794	0.3364
3	0.609	0.492	+0.496	0.3114	2	1.157	0.986	+0.895	0.3355
4	0.668	0.490	+0.594	0.2684	3	1.180	0.988	+0.993	0.3266
5	0.748	0.489	+0.686	0.2263	4	1.232	0.985	+1.091	0.3048
					5	1.298	0.981	+1.183	0.2798
1	0.777	0.655	+0.497	0.3301					
2	0.781	0.657	+0.603	0.3288	1	1.343	1.148	+0.985	0.3379
3	0.811	0.656	+0.691	0.3099	2	1.352	1.149	+1.078	0.3351
4	0.890	0.658	+0.796	0.2709	3	1.370	1.148	+1.180	0.3277
5	0.961	0.660	+0.890	0.2422	4	1.441	1.149	+1.276	0.3043

Mr. Nelles.

In his 6th and last article Bazin discusses the flow over submerged weirs, with sloping faces and crests of different widths and forms, and gives the results of 11 series, consisting of 724 individual observations. The methods used were similar to those already described for weirs of squared timbers (beam weirs). Bazin says of these experiments:

"It now remains to terminate that which concerns the flow over weirs with sloping faces by describing the experiments made with water retained below the weirs.

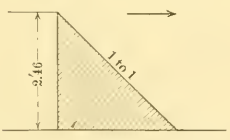
"They form 11 series, three on weirs with sharp crest, seven on weirs with crest 0.20 m. (0.66 ft.) wide, one on weirs with crest 0.40 m. (1.32 ft.) wide."

Bazin's Experiments.—Series Nos. 195 to 197.

Nappe wetted underneath, with water retained in pool below weir.

Description of weirs under observation: Height, 0.75 m. = 2.46 ft. Length, 2.0 m. = 6.56 ft. Crest sharp. Upper face vertical, lower face sloping.

Dimensions of standard weir used for measuring the discharge: Height, 1.135 m. = 3.72 ft. Length, 2.0 m. = 6.56 ft.

(1)	(2)	(3)	(4)	(5)
Number of experiment.	Observed Heads in Feet. On the weir under consideration. (h)	Observed Heads in Feet. On the standard weir of comparison. (H)	Height of the retained water with reference to crest. + = above. − = below. (h_1)	Coefficients of Discharge. For the Bazin formula. (m)
Bazin's Series No. 195. 				
1	0.310	0.331	−0.196	0.4785
2	0.318	0.326	+0.007	0.4482
3	0.337	0.328	+0.103	0.4163
4	0.383	0.331	+0.198	0.3492
5	0.419	0.331	+0.296	0.3046
1	0.452	0.490	−0.396	0.4808
2	0.461	0.493	−0.194	0.4726
3	0.465	0.489	−0.004	0.4612
4	0.488	0.491	+0.100	0.4315
5	0.510	0.497	+0.198	0.4131
6	0.533	0.495	+0.199	0.3830
7	0.556	0.500	+0.297	0.3655
8	0.594	0.490	+0.403	0.3216
1	0.605	0.654	−0.592	0.4800
2	0.600	0.653	−0.391	0.4842
3	0.610	0.654	−0.200	0.4746
4	0.620	0.654	+0.006	0.4619
5	0.634	0.660	+0.100	0.4526
6	0.650	0.653	+0.203	0.4288
7	0.674	0.654	+0.294	0.4077
8	0.702	0.660	+0.297	0.3886
9	0.732	0.656	+0.403	0.3617
10	0.806	0.656	+0.591	0.3130
1	0.743	0.819	−0.793	0.4917
2	0.750	0.817	−0.393	0.4837
3	0.746	0.817	−0.196	0.4873
4	0.758	0.812	−0.002	0.4722
5	0.792	0.815	+0.198	0.4442
6	0.844	0.815	+0.295	0.4041
7	0.863	0.813	+0.392	0.3897
8	0.930	0.814	+0.596	0.3489
9	1.024	0.818	+0.793	0.3041
1	0.885	0.976	−0.788	0.4946
2	0.886	0.973	−0.385	0.4906
3	0.901	0.988	−0.199	0.4905
4	0.908	0.979	+0.002	0.4783
5	0.941	0.984	+0.208	0.4567
6	1.016	0.982	+0.402	0.4048

Mr. Nelles.

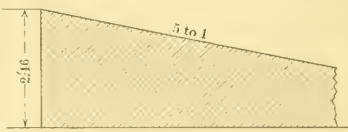
(1)	(2) h	(3) H	(4) h_1	(5) m
7	1.021	0.983	+0.402	0.4022
8	1.070	0.978	+0.600	0.3734
9	1.144	0.982	+0.795	0.3390
10	1.241	0.986	+0.986	0.3019
1	1.042	1.147	−0.582	0.4935
2	1.041	1.148	−0.389	0.4944
3	1.051	1.156	−0.197	0.4931
4	1.058	1.150	0.000	0.4848
5	1.095	1.152	+0.208	0.4614
6	1.171	1.149	+0.406	0.4154
7	1.212	1.144	+0.593	0.3919
8	1.217	1.146	+0.599	0.3904
9	1.280	1.156	+0.792	0.3664
10	1.355	1.149	+0.986	0.3336
1	1.189	1.317	−0.396	0.4994
2	1.190	1.311	−0.195	0.4958
3	1.226	1.313	+0.004	0.4753
4	1.242	1.313	+0.197	0.4658
5	1.309	1.307	+0.393	0.4274
6	1.358	1.312	+0.598	0.4070
7	1.412	1.314	+0.790	0.3847
1	1.336	1.474	−0.192	0.4970
2	1.345	1.475	+0.004	0.4926
3	1.409	1.474	+0.204	0.4599
4	1.449	1.467	+0.388	0.4371

Bazin's Series No. 196.

1	0.327	0.326	−0.200	0.4312
2	0.328	0.325	−0.006	0.4286
3	0.342	0.326	+0.097	0.4036
4	0.360	0.323	+0.189	0.3702
5	0.407	0.330	+0.289	0.3167
6	0.468	0.323	+0.392	0.2486
1	0.484	0.486	−0.200	0.4319
2	0.485	0.491	−0.001	0.4353
3	0.495	0.491	+0.097	0.4227
4	0.507	0.492	+0.189	0.4081
5	0.533	0.487	+0.292	0.3753
6	0.570	0.491	+0.392	0.3423
7	0.637	0.487	+0.493	0.2857
1	0.645	0.656	+0.001	0.4364
2	0.655	0.649	+0.194	0.4202
3	0.664	0.654	+0.207	0.4162
4	0.680	0.653	+0.296	0.4004
5	0.700	0.653	+0.395	0.3838
6	0.743	0.659	+0.492	0.3558
7	0.801	0.654	+0.589	0.3128
8	0.865	0.657	+0.688	0.2816
1	0.804	0.819	0.000	0.4371
2	0.812	0.819	+0.197	0.4316
3	0.824	0.820	+0.287	0.4215
4	0.841	0.817	+0.385	0.4078
5	0.873	0.818	+0.491	0.3863
6	0.902	0.819	+0.590	0.3676
7	0.982	0.820	+0.693	0.3240
8	1.029	0.819	+0.781	0.3020
1	0.962	0.985	+0.198	0.4414
2	0.985	0.982	+0.392	0.4235
3	1.005	0.985	+0.493	0.4126
4	1.029	0.981	+0.586	0.3963
5	1.065	0.984	+0.691	0.3789

(1)	(2) h	(3) H	(4) h_1	(5) m
6	1.146	0.983	+0.785	0.3383
7	1.196	0.983	+0.886	0.3175
8	1.239	0.981	+0.983	0.2998
1	1.119	1.152	+0.195	0.4460
2	1.131	1.150	+0.386	0.4374
3	1.147	1.143	+0.490	0.4248
4	1.164	1.146	+0.594	0.4153
5	1.199	1.146	+0.697	0.3986
6	1.230	1.149	+0.789	0.3851
7	1.281	1.150	+0.893	0.3631
8	1.358	1.147	+0.980	0.3322
9	1.414	1.153	+1.091	0.3144
1	1.281	1.313	+0.389	0.4443
2	1.311	1.311	+0.590	0.4284
3	1.330	1.306	+0.682	0.4165
4	1.357	1.311	+0.783	0.4065
5	1.390	1.313	+0.880	0.3928
6	1.434	1.314	+0.977	0.3757

Bazin's Series No. 197.

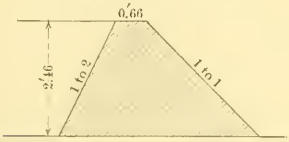


1	0.355	0.325	+0.195	0.3793
2	0.384	0.325	+0.295	0.3373
3	0.454	0.326	+0.397	0.2680
4	0.595	0.327	+0.494	0.2102
1	0.527	0.491	+0.294	0.3859
2	0.552	0.494	+0.397	0.3620
3	0.595	0.491	+0.493	0.3210
4	0.675	0.491	+0.594	0.2655
5	0.748	0.490	+0.691	0.2268
1	0.698	0.655	+0.403	0.3876
2	0 711	0.655	+0.494	0.3756
3	0.750	0.652	+0.589	0.3413
4	0.792	0.653	+0 694	0.3185
5	0.894	0.653	+0.795	0.2663
6	0.967	0.655	+0.887	0.2369
1	0.862	0.815	+0.494	0.3919
2	0.876	0.812	+0.596	0.3807
3	0.910	0.812	+0.693	0.3594
4	0.956	0.810	+0.791	0.3317
5	1.018	0.818	+0.888	0.3067
6	1.047	0.809	+0.993	0.2639
7	1.192	0.815	+1.092	0.2409
1	1.038	0.985	+0.600	0.3946
2	1.042	0.976	+0.690	0.3874
3	1.077	0.986	+0.792	0.3741
4	1.113	0.977	+0.885	0.3513
5	1.170	0.983	+0.988	0.3278
6	1.251	0.981	+1.084	0.2967
7	1.323	0.986	+1.176	0.2741
1	1.216	1.149	+0.792	0.3921
2	1.232	1.140	+0.882	0.3798
3	1.278	1.146	+0.995	0.3622
4	1.326	1.145	+1.091	0.3424
5	1.372	1.146	+1.180	0.3256
6	1.462	1.146	+1.279	0.2962
1	1.374	1.303	+0.882	0.3956
2	1.404	1.311	+0.985	0.3861
3	1.429	1.304	+1.082	0.3735
4	1.478	1.311	+1.182	0.3563

BAZIN'S EXPERIMENTS.—SERIES NOS. 198 TO 208. Mr. Nelles

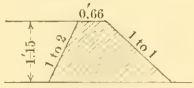
Nappe wetted underneath, with water retained in pool below weir. Description of weirs under observation: Height 0.75 m. = 2.46 ft. 0.35 m. = 1.15 ft. Length 2.0 m. = 6.56 ft. Crest 0.20 m. = 0.66 ft. wide. Upper and lower faces inclined.

Bazin's Series No. 198.



(1)	(2) h	(3) H	(4) h_1	(5) m
1	0.370	0.327	+0.098	0.3602
2	0.380	0.332	+0.144	0.3549
3	0.379	0.336	+0.147	0.3500
4	0.376	0.328	+0.197	0.3529
5	0.383	0.326	+0.295	0.3402
6	0.431	0.328	+0.394	0.2856
7	0.520	0.328	+0.492	0.2156
1	0.543	0.497	+0.197	0.3757
2	0.536	0.496	+0.198	0.3804
3	0.539	0.495	+0.240	0.3758
4	0.555	0.501	+0.289	0.3650
5	0.559	0.495	+0.403	0.3559
6	0.590	0.496	+0.492	0.3305
7	0.661	0.492	+0.592	0.2743
1	0.682	0.656	+0.198	0.4097
2	0.711	0.660	+0.395	0.3802
3	0.691	0.657	+0.397	0.3933
4	0.698	0.656	+0.438	0.3872
5	0.723	0.661	+0.509	0.3717
6	0.743	0.658	+0.589	0.3546
7	0.806	0.655	+0.687	0.3115
8	0.895	0.657	+0.789	0.2675
9	0.973	0.659	+0.888	0.2372
1	0.825	0.823	+0.299	0.4231
2	0.831	0.819	+0.395	0.4161
3	0.854	0.823	+0.488	0.4018
4	0.839	0.818	+0.491	0.4088
5	0.864	0.824	+0.543	0.3962
6	0.845	0.822	+0.544	0.4074
7	0.868	0.818	+0.591	0.3888
8	0.858	0.822	+0.592	0.3982
9	0.888	0.818	+0.688	0.3756
10	0.963	0.820	+0.788	0.3331
11	1.035	0.823	+0.879	0.3015
12	1.115	0.822	+0.994	0.2694
1	0.954	0.981	+0.300	0.4432
2	0.954	0.976	+0.395	0.4401
3	0.962	0.980	+0.490	0.4369
4	0.973	0.986	+0.592	0.4335
5	0.995	0.982	+0.598	0.4174
6	0.985	0.987	+0.635	0.4266
7	1.011	0.986	+0.692	0.4103
8	1.039	0.985	+0.789	0.3929
9	1.119	0.988	+0.884	0.3536
10	1.196	0.985	+0.983	0.3179
11	1.192	0.985	+0.987	0.3193
12	1.258	0.989	+1.083	0.2969
1	1.092	1.148	+0.397	0.4601
2	1.092	1.144	+0.496	0.4575
3	1.105	1.149	+0.593	0.4523
4	1.115	1.148	+0.689	0.4458
5	1.137	1.144	+0.691	0.4307
6	1.131	1.147	+0.745	0.4355
7	1.153	1.148	+0.788	0.4231
8	1.189	1.149	+0.884	0.4057
9	1.281	1.148	+0.979	0.3612
10	1.354	1.146	+1.087	0.3378
11	1.410	1.148	+1.182	0.3131
1	1.226	1.312	+0.494	0.4738
2	1.227	1.310	+0.593	0.4714
3	1.239	1.313	+0.689	0.4664
4	1.268	1.311	+0.740	0.4561
5	1.248	1.313	+0.741	0.4620
6	1.281	1.313	+0.785	0.4433
7	1.259	1.311	+0.786	0.4541
8	1.288	1.310	+0.835	0.4383
9	1.291	1.316	+0.841	0.4397
10	1.306	1.314	+0.888	0.4314
11	1.373	1.314	+0.983	0.4001
12	1.446	1.309	+1.083	0.3685
1	1.362	1.475	+0.594	0.4816
2	1.376	1.471	+0.784	0.4744
3	1.424	1.477	+0.887	0.4529
4	1.418	1.476	+0.887	0.4560

Bazin's Series No. 199.

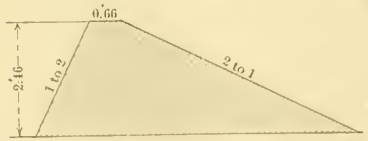


(1)	(2) h	(3) H	(4) h_1	(5) m
1	0.375	0.327	−0.190	0.3516
2	0.376	0.331	+0.001	0.3560
3	0.378	0.327	+0.178	0.3474
4	0.380	0.326	+0.276	0.3446
5	0.419	0.330	+0.376	0.3083
6	0.489	0.320	+0.460	0.2300
7	0.581	0.332	+0.561	0.1868
1	0.536	0.500	+0.002	0.3855
2	0.535	0.493	+0.177	0.3779
3	0.537	0.497	+0.273	0.3800
4	0.555	0.501	+0.366	0.3665
5	0.566	0.490	+0.460	0.3453
6	0.622	0.495	+0.565	0.3031
7	0.696	0.493	+0.649	0.2552
1	0.670	0.655	+0.002	0.4101
2	0.676	0.656	+0.176	0.4060
3	0.674	0.656	+0.273	0.4084
4	0.688	0.656	+0.373	0.3950
5	0.697	0.651	+0.466	0.3842
6	0.726	0.656	+0.565	0.3651
7	0.748	0.656	+0.644	0.3491

Mr. Nelles.

(1)	(2) h	(3) H	(4) h_1	(5) m
8	0.824	0.657	+0.745	0.3023
9	0.903	0.655	+0.838	0.2627
10	0.989	0.660	+0.933	0.2314
1	0.806	0.821	+0.095	0.4370
2	0.799	0.811	+0.271	0.4344
3	0.809	0.810	+0.367	0.4259
4	0.827	0.821	+0.464	0.4202
5	0.834	0.823	+0.504	0.4161
6	0.839	0.821	+0.519	0.4103
7	0.848	0.820	+0.568	0.4035
8	0.855	0.815	+0.681	0.3949
9	0.880	0.813	+0.736	0.3771
10	0.947	0.817	+0.837	0.3491
11	1.018	0.811	+0.923	0.3019
12	1.094	0.814	+1.013	0.2725
1	0.932	0.983	+0.279	0.4608
2	0.941	0.985	+0.367	0.4557
3	0.944	0.980	+0.465	0.4495
4	0.960	0.980	+0.557	0.4382
5	0.976	0.981	+0.646	0.4281
6	0.993	0.976	+0.740	0.4153
7	1.019	0.979	+0.837	0.4003
8	1.082	0.978	+0.932	0.3659
9	1.146	0.976	+1.016	0.3350
10	1.220	0.979	+1.108	0.3057
11	1.305	0.978	+1.212	0.2758
1	1.064	1.150	+0.370	0.4789
2	1.070	1.147	+0.461	0.4731
3	1.085	1.159	+0.517	0.4710
4	1.080	1.147	+0.555	0.4668
5	1.098	1.149	+0.653	0.4559
6	1.111	1.149	+0.735	0.4480
7	1.135	1.150	+0.835	0.4347
8	1.154	1.151	+0.921	0.4357
9	1.200	1.143	+1.015	0.3955
10	1.274	1.146	+1.108	0.3636
11	1.360	1.150	+1.208	0.3316
1	1.190	1.309	+0.461	0.4934
2	1.209	1.318	+0.563	0.4865
3	1.213	1.305	+0.647	0.4770
4	1.223	1.293	+0.749	0.4648
5	1.253	1.314	+0.842	0.4594
6	1.272	1.316	+0.934	0.4499
7	1.284	1.315	+1.012	0.4426
8	1.341	1.320	+1.113	0.4172
9	1.420	1.311	+1.206	0.3792
1	1.339	1.469	+0.655	0.4970
2	1.349	1.478	+0.736	0.4911
3	1.355	1.475	+0.837	0.4859
4	1.373	1.475	+0.917	0.4765
5	1.389	1.474	+1.019	0.4679
6	1.419	1.476	+1.115	0.4545
7	1.474	1.476	+1.205	0.4298
8	1.531	1.472	+1.291	0.4036

Bazin's Series No. 200.



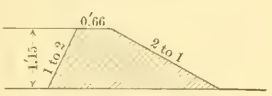
1	0.386	0.335	+0.093	0.3480
2	0.385	0.330	+0.189	0.3461

(1)	(2) h	(3) H	(4) h_1	(5) m
3	0.387	0.334	+0.234	0.3456
4	0.383	0.329	+0.235	0.3448
5	0.387	0.329	+0.282	0.3382
6	0.392	0.326	+0.292	0.3278
7	0.405	0.331	+0.331	0.3202
8	0.437	0.332	+0.391	0.2850
9	0.518	0.326	+0.494	0.2146
10	0.616	0.330	+0.591	0.1696
1	0.550	0.502	+0.092	0.3724
2	0.544	0.497	+0.189	0.3726
3	0.544	0.494	−0.255	0.3701
4	0.544	0.492	+0.291	0.3679
5	0.557	0.496	+0.384	0.3595
6	0.561	0.562	+0.387	0.3610
7	0.567	0.494	+0.435	0.3473
8	0.569	0.493	+0.437	0.3447
9	0.569	0.497	+0.457	0.3489
10	0.590	0.496	+0.490	0.3299
11	0.584	0.495	+0.494	0.3333
12	0.602	0.499	+0.533	0.3229
13	0.666	0.496	+0.587	0.2743
14	0.749	0.495	+0.682	0.2294
15	0.839	0.491	+0.787	0.1908
1	0.699	0.661	+0.189	0.3904
2	0.703	0.664	+0.285	0.3901
3	0.704	0.661	+0.386	0.3865
4	0.714	0.659	+0.487	0.3760
5	0.722	0.666	+0.539	0.3756
6	0.728	0.657	+0.566	0.3645
7	0.733	0.655	+0.590	0.3590
8	0.734	0.659	+0.590	0.3610
9	0.751	0.658	+0.634	0.3485
10	0.759	0.657	+0.635	0.3421
11	0.802	0.657	+0.687	0.3144
12	0.894	0.655	+0.789	0.2662
13	0.970	0.655	+0.886	0.2351
14	1.050	0.657	+0.977	0.1799
1	0.840	0.821	+0.187	0.4102
2	0.837	0.820	+0.294	0.4112
3	0.837	0.815	+0.389	0.4080
4	0.842	0.815	+0.491	0.4035
5	0.852	0.815	+0.589	0.3977
6	0.869	0.812	+0.687	0.3818
7	0.880	0.819	+0.686	
8	0.888	0.817	+0.731	0.3744
9	0.903	0.819	+0.741	0.3665
10	0.948	0.823	+0.787	0.3433
11	0.908	0.819	+0.788	0.3629
12	0.931	0.818	+0.821	0.3492
13	1.002	0.814	+0.842	0.3103
14	1.032	0.809	+0.883	0.2944
15	1.115	0.815	+0.979	0.2653
16	1.199	0.820	+1.092	0.2398
17	1.274	0.819	+1.183	0.2188
1	0.978	0.982	+0.290	0.4279
2	0.978	0.979	+0.387	0.4252
3	0.983	0.982	+0.493	0.4244
4	0.987	0.980	+0.588	0.4199
5	0.997	0.977	+0.686	0.4124
6	1.012	0.976	+0.785	0.4023
7	1.031	0.982	+0.826	0.3947
8	1.041	0.978	+0.842	0.3870
9	1.044	0.974	+0.880	0.3828
10	1.082	0.982	+0.881	0.3672
11	1.067	0.979	+0.925	0.3732
12	1.144	0.978	+0.933	0.3358
13	1.193	0.976	+0.983	0.3144
14	1.236	0.971	+1.048	0.2959
15	1.262	0.982	+1.077	0.2913
16	1.339	0.979	+1.176	0.2655

Mr. Nelles.

(1)	(2) h	(3) H	(4) h_1	(5) m
1	1.120	1.148	+0.295	0.4422
2	1.118	1.148	+0.389	0.4432
3	1.119	1.146	+0.509	0.4408
4	1.125	1.148	+0.590	0.4385
5	1.123	1.141	+0.689	0.4356
6	1.144	1.149	+0.781	0.4286
7	1.158	1.141	+0.884	0.4159
8	1.173	1.143	+0.928	0.4093
9	1.236	1.141	+0.983	0.3769
10	1.189	1.140	+0.984	0.3992
11	1.190	1.136	+1.032	0.3516
12	1.209	1.135	+1.039	0.3865
13	1.240	1.147	+1.067	0.3787
14	1.337	1.146	+1.075	0.3377
15	1.410	1.148	+1.181	0.3125
1	1.250	1.300	+0.394	0.4528
2	1.250	1.296	+0.487	0.4507
3	1.254	1.297	+0.591	0.4496
4	1.256	1.301	+0.682	0.4501
5	1.263	1.297	+0.786	0.4440
6	1.274	1.296	+0.888	0.4380
7	1.299	1.297	+0.951	0.4261
8	1.325	1.296	+0.981	0.4128
9	1.379	1.296	+1.081	0.3892
10	1.448	1.296	+1.084	0.3636
11	1.453	1.300	+1.133	0.4011
12	1.466	1.305	+1.174	0.3844

Bazin's Series No. 201.



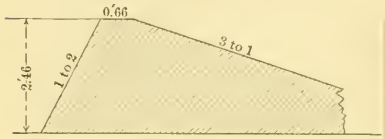
1	0.370	0.327	+0.141	0.3600
2	0.379	0.332	+0.178	0.3566
3	0.389	0.342	+0.231	0.3560
4	0.388	0.340	+0.234	0.3532
5	0.385	0.333	+0.277	0.3462
6	0.389	0.325	+0.328	0.3297
7	0.413	0.330	+0.365	0.3085
8	0.491	0.343	+0.462	0.2522
9	0.530	0.330	+0.509	0.2119
1	0.535	0.496	+0.180	0.3817
2	0.548	0.509	+0.232	0.3822
3	0.442	0.501	+0.277	0.3799
4	0.539	0.494	+0.326	0.3747
5	0.546	0.502	+0.364	0.3765
6	0.551	0.501	+0.418	0.3691
7	0.547	0.492	+0.421	0.3644
8	0.558	0.491	+0.461	0.3525
9	0.584	0.501	+0.512	0.3384
10	0.622	0.502	+0.565	0.3094
11	0.709	0.500	+0.657	0.2527
12	0.791	0.495	+0.754	0.2112
1	0.687	0.666	+0.230	0.4051
2	0.692	0.669	+0.274	0.4034
3	0.684	0.656	+0.320	0.3987
4	0.682	0.659	+0.365	0.4037
5	0.692	0.661	+0.461	0.3964
6	0.694	0.650	+0.516	0.3856
7	0.712	0.655	+0.592	0.3744
8	0.708	0.654	+0.565	0.3772
9	0.726	0.660	+0.613	0.3687
10	0.730	0.665	+0.614	0.3692
11	0.755	0.661	+0.661	0.3480

(1)	(2) h	(3) H	(4) h_1	(5) m
12	0.784	0.661	+0.700	0.3283
13	0.832	0.665	+0.748	0.3081
14	0.914	0.664	+0.849	0.2632
15	0.992	0.663	+0.940	0.2324
1	0.815	0.818	+0.273	0.4262
2	0.819	0.819	+0.327	0.4249
3	0.817	0.819	+0.367	0.4265
4	0.815	0.812	+0.469	0.4221
5	0.839	0.827	+0.561	0.4150
6	0.847	0.820	+0.656	0.4041
7	0.855	0.817	+0.705	0.3964
8	0.854	0.810	+0.711	0.3923
9	0.875	0.823	+0.747	0.3877
10	0.869	0.810	+0.747	0.3816
11	0.910	0.829	+0.790	0.3694
12	0.950	0.819	+0.842	0.3395
13	1.041	0.821	+0.934	0.2973
14	1.077	0.817	+0.984	0.2800
15	1.111	0.819	+1.026	0.2686
16	1.190	0.812	+1.126	0.2396
1	0.950	0.981	+0.325	0.4457
2	0.947	0.979	+0.371	0.4463
3	0.954	0.988	+0.460	0.4481
4	0.952	0.976	+0.507	0.4412
5	0.954	0.983	+0.559	0.4442
6	0.964	0.984	+0.663	0.4382
7	0.956	0.974	+0.746	0.4269
8	0.989	0.982	+0.792	
9	0.994	0.989	+0.794	0.4219
10	0.991	0.986	+0.791	
11	1.001	0.976	+0.839	0.4095
12	1.006	0.983	+0.848	0.4102
13	1.035	0.986	+0.884	0.3955
14	1.024	0.990	+0.888	0.4046
15	1.077	0.987	+0.931	0.3739
16	1.075	0.988	+0.933	
17	1.121	0.983	+0.983	0.3335
18	1.168	0.979	+1.030	0.3269
19	1.237	0.984	+1.114	0.3016
20	1.320	0.983	+1.219	0.2732
1	1.080	1.146	+0.467	0.4661
2	1.085	1.143	+0.561	0.4602
3	1.085	1.146	+0.658	0.4624
4	1.093	1.141	+0.755	0.4541
5	1.108	1.145	+0.837	0.4474
6	1.126	1.152	+0.885	0.4419
7	1.129	1.142	+0.936	0.4334
8	1.158	1.153	+0.991	0.4234
9	1.164	1.151	+0.992	0.4192
10	1.205	1.146	+1.029	0.3953
11	1.244	1.150	+1.071	0.3789
12	1.292	1.144	+1.118	0.3546
13	1.373	1.146	+1.222	0.3246
1	1.204	1.300	+0.567	0.4791
2	1.212	1.306	+0.655	0.4781
3	1.223	1.313	+0.739	0.4757
4	1.221	1.299	+0.841	0.4690
5	1.234	1.303	+0.939	0.4634
6	1.244	1.293	+0.987	0.4531
7	1.261	1.305	+1.035	0.4503
8	1.281	1.305	+1.077	0.4393
9	1.320	1.303	+1.120	0.4146
10	1.377	1.303	+1.170	0.3934
11	1.428	1.294	+1.222	0.3684
12	1.459	1.297	+1.264	0.3588
13	1.503	1.304	+1.314	0.3455
1	1.333	1.461	+0.657	0.4911
2	1.335	1.461	+0.742	0.4905

Mr. Nelles.

(1)	(2) h	(3) H	(4) h_1	(5) m
3	1.337	1.460	+0.842	0.4892
4	1.343	1.458	+0.933	0.4846
5	1.370	1.470	+1.029	0.4763
6	1.394	1.468	+1.119	0.4626
7	1.405	1·462	+1.171	0.4516
8	1.433	1.462	+1.212	0.4419
9	1.494	1.465	+1.263	0.4153
10	1.541	1.464	+1.313	0.3961

Bazin's Series No. 202.

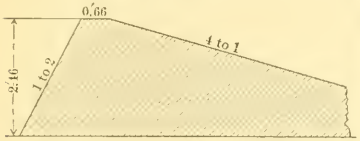


1	0.380	0.330	+0.192	0.3523
2	0.383	0.328	+0.290	0.3442
3	0.408	0.339	+0.345	0.3280
4	0.427	0.330	+0.394	0.2945
5	0.471	0.336	+0.439	0.2602
6	0.513	0.329	+0.487	0.2227
1	0.548	0.494	+0.290	0.3674
2	0.552	0.493	+0.389	0.3614
3	0.570	0.493	+0.487	0.3447
4	0.610	0.503	+0.547	0.3211
5	0.616	0.504	+0.598	0.2952
6	0.684	0.506	+0.637	0.2725
7	0.731	0.500	+0.687	0.2422
1	0.703	0.656	+0.286	0.3838
2	0.704	0.657	+0.388	0.3833
3	0.707	0.655	+0.479	0.3792
4	0.738	0.665	+0.588	0.3644
5	0.756	0.656	+0.673	0.3448
6	0.832	0.666	+0.748	0.3048
7	0.861	0.661	+0.794	0.2865
8	0.923	0.665	+0.842	0.2609

(1)	(2) h	(3) H	(4) h_1	(5) m
9	0.946	0.663	+0.882	0.2498
1	0.853	0.824	−0.190	0.4024
2	0.853	0.820	+0.006	0.4018
3	0.853	0.823	+0.200	0.4040
4	0.854	0.822	+0.395	0.4017
5	0.858	0.824	+0.485	0.4006
6	0.870	0.827	+0.600	0.3943
7	0.884	0.823	+0.686	0.3829
8	0.910	0.825	+0.788	0.3679
9	0.978	0.820	+0.869	0.3268
10	1.088	0.824	+0.983	0.2808
11	1.176	0.832	+1.092	0.2529
1	1.001	0.987	+0.282	0.4176
2	1.002	0.986	+0.388	0.4159
3	1.002	0.984	+0.477	0.4151
4	1.012	0.993	+0.584	0.4141
5	1.010	0.983	+0.671	0.4086
6	1.026	0.989	+0.787	0.4035
7	1.055	0.991	+0.877	0.3877
8	1.092	0.991	+0.966	0.3693
9	1.109	0.988	+0.987	0.3590
10	1.198	0.991	+1.064	0.3203
11	1.283	0.992	+1.184	0.2897
12	1.375	0.993	+1.269	0.2617
1	1.152	1.150	+0.491	0.4256
2	1.151	1.150	+0.590	0.4271
3	1.151	1.156	+0.687	0.4249
4	1.171	1.161	+0.783	0.4224
5	1.182	1.151	+0.897	0.4107
6	1.201	1.154	+0.975	0.4028
7	1.247	1.152	+1.091	0.3795
8	1.322	1.158	+1.168	0.3504
9	1.454	1.157	+1.286	0.3035
1	1.295	1.317	+0.590	0.4392
2	1.295	1.314	+0.688	0.4379
3	1.302	1.317	+0.784	0.4357
4	1.311	1.316	+0.885	0.4310
5	1.326	1.320	+0.976	0.4255
6	1.342	1.314	+1.084	0.4156
7	1.385	1.323	+1.180	0.3998
8	1.443	1.316	+1.279	0.3738

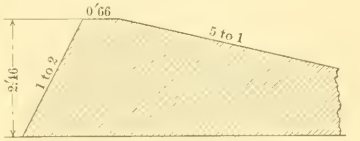
Mr. Nelles.

Bazin's Series No. 203.



(1)	(2) h	(3) H	(4) h_1	(5) m
1	0.380	0.329	+0.186	0.3505
2	0.380	0.327	+0.284	0.3465
3	0.391	0.329	+0.232	0.3356
4	0.417	0.327	+0.385	0.3013
5	0.453	0.328	+0.431	0.2656
6	0.501	0.326	+0.484	0.2277
1	0.546	0.494	+0.291	0.3695
2	0.544	0.490	+0.388	0.3661
3	0.556	0.490	+0.485	0.3539
4	0.580	0.492	+0.533	0.3351
5	0.619	0.488	+0.584	0.2992
6	0.668	0.489	+0.632	0.2685
7	0.714	0.492	+0.684	0.2449
1	0.707	0.655	+0.289	0.3804
2	0.716	0.661	+0.380	0.3781
3	0.705	0.656	+0.391	0.3829
4	0.713	0.657	+0.491	0.3768
5	0.717	0.657	+0.588	0.3734
6	0.754	0.658	+0.684	0.3482
7	0.798	0.653	+0.730	0.3152
8	0.835	0.658	+0.780	0.2974
9	0.885	0.659	+0.831	0.2743
10	0.930	0.656	+0.886	0.2523
1	0.856	0.821	+0.293	0.3996
2	0.861	0.823	+0.393	0.3985
3	0.852	0.815	+0.490	0.3987
4	0.870	0.825	+0.591	0.3939
5	0.874	0.820	+0.683	0.3870
6	0.899	0.825	+0.785	0.3746
7	0.949	0 819	+0.881	0.3416
8	1.049	0.825	+0.981	0.2969
9	1.147	0.819	+1.078	0.2568
1	1.016	0.988	+0.482	0.4092
2	1.015	0.984	+0.592	0.4075
3	1.019	0.983	+0.680	0.4043
4	1.026	0.983	+0.780	0.4001
5	1.045	0.982	+0.878	0.3886
6	1.085	0.985	+0.983	0.3690
7	1.166	0.984	+1.072	0.3306
8	1.284	0.985	+1.182	0.2864
9	1.365	0.885	+1.274	0.2612
1	1.170	1.151	+0.578	0.4170
2	1.158	1.141	+0.689	0.4173
3	1.179	1.154	+0.775	0.4137
4	1.180	1.150	+0.881	0.4108
5	1.196	1.150	+0.976	0.4030
6	1.222	1.147	+1.072	0.3888
7	1.288	1.150	+1.176	0.3603
8	1.330	1.150	+1.273	0.3428
1	1.321	1.315	+0.682	0.4256
2	1.322	1.313	+0.784	0.4242
3	1.326	1.313	+0.879	0.4223
4	1.337	1.316	+0.980	0.4182
5	1.348	1.316	+1.068	0.4134
6	1.382	1.318	+1.181	0.3990
7	1.422	1.314	+1.278	0.3806

Bazin's Series No. 204.



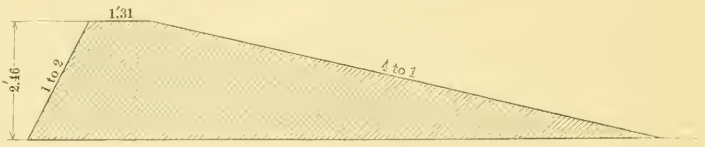
(1)	(2) h	(3) H	(4) h_1	(5) m
1	0.384	0.330	+0.186	0.3459
2	0.383	0.329	+0.294	0.3457
3	0.390	0.330	+0.338	0.3380
4	0.430	0.329	+0.389	0.2897
5	0.450	0.330	+0.435	0·2718
6	0.510	0.328	+0.493	0.2237
1	0.545	0.487	+0.287	0.3620
2	0.556	0.495	+0.394	0.3609
3	0.556	0.489	+0.487	0.3537
4	0.582	0.490	+0.535	0.3310
5	0.588	0.496	+0.538	0.3326
6	0.622	0.486	+0.589	0.2966
7	0.658	0.492	+0.632	0.2764
8	0.720	0.486	+0.681	0.2380
1	0.714	0.659	+0.494	0.3774
2	0.723	0.657	+0.587	0.3693
3	0.765	0.658	+0.671	0.3410
4	0.786	0.662	+0.734	0.3288
5	0.833	0.658	+0.775	0.2992
6	0.880	0.663	+0.830	0.2785
7	0.929	0.676	+0.890	0.2642
1	0.871	0.827	+0.297	0.3930
2	0.870	0.822	+0.383	0.3903
3	0.883	0.833	+0.492	0.3901
4	0.875	0.822	+0.579	0.3873
5	0.894	0.836	+0.692	0.3848
6	0.900	0.822	+0.779	0.3711
7	0.962	0.832	+0.891	0.3422
8	1.041	0.822	+0.969	0.2988
9	1.145	0.832	+1.086	0.2638
1	1.026	0.986	+0.586	0.4022
2	1.025	0.984	+0.688	0.4012
3	1.025	0.978	+0.786	0.3981
4	1.049	0.982	+0.885	0.3866
5	1.070	0.978	+0.978	0.3731
6	1.158	0.983	+1.077	0.3334
7	1.260	0.983	+1.172	0.2940
8	1.349	0.982	+1.273	0.2647
1	1.175	1.148	+0.591	0.4129
2	1.183	1.147	+0.686	0.4080
3	1.182	1.150	+0.784	0.4101
4	1.188	1.144	+0.879	0.4041
5	1.211	1.153	+0.986	0.3971
6	1.227	1.150	+1.072	0.3880
7	1.276	1.146	+1.180	0.3632
8	1.380	1.148	+1.270	0.3243
1	1.329	1.310	+0.689	0.4190
2	1.338	1.313	+0.782	0.4163
3	1.337	1.312	+0.887	0.4166
4	1.347	1.312	+0.976	0.4119
5	1.353	1.305	+1.078	0.4065
6	1.380	1.313	+1.175	0.3975
7	1.421	1.310	+1.276	0.3791

Mr. Nelles.

Bazin's Experiments.—Series No. 205.

Nappe wetted underneath, with water retained in pool below weir. Description of weir under observation: Height, 0.75 m. = 2.46 ft. Length, 2.0 m. = 6.56 ft. Crest, 0.40 m. = 1.31 ft. wide. Both faces inclined.

Bazin's Series No. 205.



(1)	(2) h	(3) H	(4) h_1	(5) m	(1)	(2) h	(3) H	(4) h_1	(5) m
1	0.385	0.330	+0.305	0.3438	5	0.922	0.815	+0.788	0.3544
2	0.393	0.332	+0.354	0.3368	6	0.956	0.819	+0.884	0.3375
3	0.418	0.328	+0.393	0.3010	7	1.029	0.814	+0.986	0.2997
4	0.461	0.327	+0.441	0.2501	8	1.133	0.819	+1.084	0.2619
5	0.512	0.329	+0.491	0.2230	9	1.226	0.814	+1.183	0.2309
1	0.561	0.491	+0.386	0.3505	1	1.075	0.986	+0.694	0.3752
2	0.570	0.493	+0.491	0.3444	2	1.075	0.983	+0.782	0.3733
3	0.593	0.493	+0.538	0.3240	3	1.091	0.988	+0.886	0.3681
4	0.622	0.496	+0.593	0.3045	4	1.106	0.984	+0.975	0.3581
5	0.675	0.493	+0.642	0.2662	5	1.159	0.987	+1.081	0.3355
6	0.709	0.495	+0.689	0.2503	6	1.247	0.985	+1.178	0.2997
					7	1.345	0.987	+1.286	0.2684
1	0.749	0.655	+0.591	0.3482					
2	0.749	0.660	+0.639	0.3531	1	1.238	1.147	+0.886	0.3813
3	0.768	0.652	+0.691	0.3335	2	1.254	1.151	+0.981	0.3767
4	0.794	0.658	+0.741	0.3220	3	1.258	1.143	+1.081	0.3697
5	0.834	0.657	+0.786	0.2980	4	1.292	1.146	+1.179	0.3568
6	0.877	0.660	+0.839	0.2789	5	1.360	1.144	+1.277	0.3292
7	0.922	0.657	+0.887	0.2568					
					1	1.399	1.311	+0.985	0.3891
1	0.900	0.816	+0.394	0.3681	2	1.413	1.311	+1.088	0.3830
2	0.900	0.818	+0.497	0.3695	3	1.426	1.312	+1.182	0.3783
3	0.904	0.816	+0.590	0.3649	4	1.449	1.312	+1.279	0.3694
4	0.908	0.822	+0.688	0.3669					

"We have already shown, in discussing the similar experiments on beam weirs, that the size of the crest completely modifies the effect of the retained water. Thus, for weirs in thin partition, the effect of the retained water is manifest before it reaches the level of the crest, while for weirs with wide crest, the effect is not noticeable until the level of the retained water is above the crest. In order to show the results of the preceding experiments in a comprehensive manner, we have prepared the following tables, which show for the same discharge and for varying elevations of the retained water, the corresponding heads on the weirs under experiment.

"Notwithstanding some anomalies, the tables show that the influence of the retained water is hardly sensible so long as its height above the crest h_1 is less than one-half the head h. When we consider Series 205, in which the crest is 0.40 m. (1.31 ft.) wide, it is seen that the influence of the retained water is not appreciable until it attains a height of two-thirds of the head."

Bazin's Experiments.

Mr. Nelles.

Table Showing the Heads Corresponding to the same Discharge over Weirs with Sharp Crests and Sloping Faces, for Various Elevations of the Surface of the Water Retained Below Weir.

Head on standard weir of comparison. (H')	Head when the discharge is not affected by the retained water.	Head, in Feet, on the Weirs under Consideration. (h.) Height of the water surface below the weirs, below or above the level of the crest. (h_1)								
		−0.198	0	+0.198	+0.394	+0.591	+0.787	+0.984	+1.181	
0.492	0.456	0.459	0.472	0.515	0.591	0.591	0.591	0.591	0.591	*Series No. 195.* Upper face vertical; lower face 1 to 1.
0.656	0.601	0.611	0.620	0.653	0.722	0.810	0.810	0.810	0.810	
0.820	0.748	0.752	0.765	0.807	0.869	0.942	1.024	1.024	1.024	
0.984	0.893	0.899	0.909	0.939	1.000	1.080	1.158	1.237	1.237	
1.148	1.040	1.043	1.060	1.096	1.151	1.217	1.286	1.352	1.352	
1.312	1.184	1.194	1.214	1.254	1.302	1.355	1.408	1.470	1.470	
0.492	0.485	0.485	0.489	0.512	0.575	0.575	0.575	0.575	0.575	*Series No. 196.* Upper face vertical; lower face 2 to 1.
0.656	0.643	0.643	0.646	0.666	0.702	0.797	0.797	0.797	0.797	
0.820	0.797	0.797	0.804	0.813	0.843	0.919	1.033	1.033	1.033	
0.984	0.948	0.948	0.948	0.964	0 987	1.037	1.136	1.250	1.250	
1.148	1.109	1.109	1.109	1.116	1.136	1.168	1.230	1.348	1.348	
1.312	1.267	1.267	1.267	1.267	1.282	1.312	1.365	1.437	1.437	
0.492	0.528	0.528	0.528	0.528	0.551	0.669	0.669	0.669	0.669	*Series No. 197.* Upper face vertical; lower face 5 to 1.
0.656	0.696	0.696	0.696	0.696	0.699	0.745	0.889	0.889	0.889	
0.820	0.863	0.863	0.863	0.863	0.863	0.886	0.961	1.076	1.076	
0.984	1.030	1.030	1.030	1.030	1.030	1.033	1.073	1.174	1.325	
1.148	1.201	1.201	1.201	1.201	1.201	1.201	1.217	1.279	1.388	
1.312	1.368	1.368	1.368	1.368	1.368	1.368	1.368	1.404	1.483	

Table Showing the Heads Corresponding to the same Discharge over Weirs with Crests 0.20 m. = 0.66 ft. wide, and Sloping Faces, for Various Elevations of the Surface of the Retained Water.

0.492	0.528	.....	.....	0.532	0.522	0.653	0.653	0.653	0.653	*Series No. 198.* Upper face inclined, 1 to 2; lower face inclined, 1 to 1.
0.656	0.682	.....	.....	0.686	0.694	0.745	0.889	0.889	0.889	
0.820	0.817	.....	.....	0.817	0.830	0.863	0.964	1.103	1.103	
0.984	0.954	.....	.....	0.954	0.961	0.981	1.053	1.188	1.188	
1.148	1.090	.....	.....	1.090	1.090	1.106	1.155	1.273	1.273	
1.312	1.217	.....	.....	1.217	1.267	1.230	1.267	1.371	1.371	
0.492	0.538	.....	.....	0.542	0.555	0.646	0.843	0.843	0.843	*Series No. 200.* Upper face inclined, 1 to 2; lower face inclined, 2 to 1.
0.656	0.686	.....	.....	0.689	0.696	0.732	0.889	1.063	1.063	
0.820	0.833	.....	.....	0.837	0.840	0.856	0.939	1.116	1.286	
0.984	0.974	.....	.....	0.974	0.981	0.987	1.030	1.178	1.361	
1.148	1.113	.....	.....	1.113	1.119	1.123	1.148	1.217	1.418	
1.312	1.257	.....	.....	1.257	1.260	1.267	1.276	1.315	1.457	
0.492	0.542	.....	.....	0.545	0.551	0.623	0.623	0.623	0.623	*Series No. 202.* Upper face inclined, 1 to 2; lower face inclined, 3 to 1.
0.656	0.693	.....	.....	0.702	0.706	0.732	0.850	0.850	0.850	
0.820	0.850	.....	.....	0.850	0.856	0.860	0.912	1.080	1.080	
0.984	1.000	.....	.....	1.000	1.000	1.004	1.024	1.109	1.276	
1.148	1.145	.....	.....	1.145	1.145	1.148	1.158	1.197	1.322	
1.312	1.289	.....	.....	1.289	1.289	1.289	1.295	1.319	1.375	
0.492	0.542	.....	.....	0.542	0.548	0.623	0.623	0.623	0.623	*Series No. 203.* Upper face inclined, 1 to 2; lower face inclined, 4 to 1.
0.656	0.706	.....	.....	0.706	0.709	0.722	0.843	0.843	0.843	
0.820	0.856	.....	.....	0.856	0.856	0.863	0.899	1.043	1.043	
0.984	1.010	.....	.....	1.010	1.010	1.014	1.027	1.090	1.276	
1.148	1.161	.....	.....	1.161	1.161	1.164	1.171	1.197	1.282	
1.312	1.312	.....	.....	1.312	1.312	1.312	1.319	1.332	1.375	
0.192	0.548	.....	.....	0.548	0.551	0.630	0.630	0.630	0.630	*Series No. 204.* Upper face inclined, 1 to 2; lower face inclined, 5 to 1.
0.656	0.706	.....	.....	0.706	0.725	0.833	0.833	0.833	0.833	
0.820	0.863	.....	.....	0.863	0.866	0.873	0.906	1.037	1.037	
0.984	1.024	.....	.....	1.024	1.024	1.030	1.090	1.263	1.263	
1.148	1.178	.....	.....	1.178	1.178	1.178	1.181	1.204	1.289	
1.312	1.332	.....	.....	1.332	1.332	1.332	1.335	1.345	1.385	

Mr. Nelles. From a consideration of the foregoing and other experiments on the flow over submerged weirs, the writer is impressed with the idea that the usual laboratory methods are to some extent defective and misleading, in that the conditions are artificial, and differ, except for the rare case of a flow into a reservoir, from those met in actual practice. In the laboratory experiments, the level of the backwater is regulated by impeding the flow away from the weir, and decreasing the velocity of departure; whereas, in practice, in the usual case of a dam in a flowing stream, the conditions are natural, and the height of the water surface below and the fall over the dam depend upon the quantity of water passing. It is not unlikely that some of the discrepancies hereafter noted are due to this cause.

The most common use to which engineers apply the theory and formulas for the flow over submerged dams is in connection with the slack-water improvement of rivers, for the determination of the effect of the dams in raising the water surface above them, and for the determination of the fall at the dams, in order to fix the height of the lock walls and the stage at which navigation over the dam itself will become possible.

Several formulas have been proposed for this purpose. That which is most generally used is based on the theory that the flow above the level of the lower pool is similar to a free discharge into air, and that the usual weir formulas for such discharge are applicable for its determination. The flow between the level of the lower pool and the crest of the dam is considered similar to that from a submerged orifice, and determined by the recognized formulas for such flow. By combining the common expressions for the two forms of flow, the following general formula (which should be corrected for the effect of the velocity of approach), is obtained:

$$Q = C' L \sqrt{2 g z}\,(h_1 + \tfrac{2}{3} z) \qquad (b)$$

in which

Q = discharge, in cubic feet per second;
C' = a coefficient;
L = crest length of dam, in feet;
z = fall at dam, $= h - h_1$;
h_1 = height of lower pool above crest of dam.

Several methods of taking into account the effect of the velocity of approach have been advanced. The simplest of these, although not the most accurate, which will be used in the present discussion, consists of increasing the head to which the velocity of flow is due, by the head to which the mean velocity of approach is due, making the quantity under the radical in Equation (b)

$$\sqrt{2 g \left(z + \frac{v^2}{2 g}\right)}$$

Mr. Nelles.

The theory of Equation (*b*) makes the coefficient C'' equal to 1.5 times the coefficient m of the free portion of the discharge.

Francis, from small-scale experiments on sharp-crested vertical weirs, makes this ratio 1.38. D'Aubisson gives it at 1.43. Weisbach accepts the value assigned by theory.

The results of a large number of Bazin's experiments, Series Nos. 200 and 201, have been applied to Formula (*b*) by the writer, giving values for the coefficient C'' of from 0.71 to 1.20, and of the ratio $\frac{C''}{m}$ of from 1.54 to 2.40. These results, which included the effect of the velocity of approach, showed, without apparent reason, such a wide variation that no satisfactory conclusions could be drawn from the investigation.

From a careful study and manipulation of the gauge records at the locks and dams on the Kentucky River, covering 65 days in February, March and April, 1886,* it has been possible to deduce approximate values for the ratio $\frac{C''}{m}$ for these dams. The dams consist of rock-filled cribs, with planked up-stream faces sloping at 3 to 1, and with vertical steps below the crest. The height of the dams is about 20 ft.

The deductions were made as follows: From the gauge records, periods of nearly uniform flow and conditions were selected, during which at least one of the dams was submerged, while one or more of the adjacent dams was not. Then, from the mean head for the selected period at the dams not submerged, a probable mean discharge was obtained by the usual weir formula. This discharge was assumed to be that at the dam under submerged flow, and, with the other mean conditions shown by the records to exist at the submerged dam, was substituted in Formula (*b*), which then gave the corresponding value for the coefficient C'. By comparing the value thus obtained with the value of the coefficient m, used in determining the free discharge, the ratio $\frac{C'}{m}$ was obtained.

In this way, three separate determinations were made. The first, covering four days, when the water was falling slowly, Dams Nos. 4 and 5 being under free flow, and Dam No. 3 submerged, giving $\frac{C'}{m} = 1.50$.

The second, covering three days, while the water was falling slowly, Dams Nos. 3 and 4 being under free flow, and Dam No. 2 submerged, giving $\frac{C'}{m} = 1.53$.

The third, covering five days, during which the river rose and fell

* Annual Report, Chief of Engineers, U. S. A., 1887, pp. 1318 and 1319.

Mr. Nelles. slowly, Dams Nos. 2 and 3 being under free flow, and Dam No. 1 submerged, giving $\frac{C'}{m} = 1.46$. The mean value of the ratio $\frac{C'}{m}$ for the 11 days being 1.49.

The depths on the crest ranged from 4 to 7.5 ft., with a mean of 5.3 ft.

Another formula, used frequently for the same purposes as Formula (*b*), is known as the Chanoine and Mary Formula.

$$Q = M L h_1 \sqrt{2 g z} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (c)$$

This is of the same form as the usual submerged orifice or sluice formulas, and, while it is applicable to conditions identical with those for which the coefficient M is known, it cannot be safely used otherwise. This is demonstrated by Edward Sawyer, M. Am. Soc. C. E., in an article,* in which he illustrates his assertion by assuming a case wherein the level of the upper pool remains constant, while that of the lower pool is lowered. For this case, the formula gives a decreasing value for Q, whereas the contrary would actually result.

The formula given by Chanoine and DeLagrene, in their memoir on the dams of the Upper Seine, is very generally used by French engineers. This formula is based on the theory that for a given discharge, the effect produced by a submerged dam, or other impediment to flow, in raising the level of the water surface above it, is represented by the difference between the heads to which the mean velocity at the dam site is due, before and after the execution of the works, multiplied by a coefficient to which is usually given a value of 1.5. This theory is expressed by the following formula:

$$z = K \left(\frac{V_1^2 - V^2}{2 g} \right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (d)$$

The writer has tested this formula by the substitution of the data taken from Bazin's Series No. 200. The resulting values for the coefficient K ranged from 0.25 to 1.45, and were so erratic that the selection of even a probable value was found practically impossible. A further test was made by using the results of the actual observations on the flow over the twelve dams on the Upper Seine.† In this way K was found to vary, without apparent reason, independent of z, from 0.30 to 2.50.

The most elaborate published account of the flow over submerged dams of practical dimensions and form is found in a paper by Mr. R. H. Rhind.‡ This paper describes and discusses in considerable detail observations made at a number of large masonry dams in India. Most of the dams have flat crests, with paved slopes. The exact dimensions in these particulars are not given. The discharge was calculated by

* *Van Nostrand's Magazine*, Vol. xxxiv, p. 176.

† *Annales des Ponts et Chaussées*, 1868.

‡ *Minutes of Proceedings*, Institute of Civil Engineers, for 1886.

Mr. Nelles

TABLE No. 22.—Dimensions of and Observed Conditions of Flow at Certain Large Masonry Dams in India, with Corresponding Coefficients of Discharge.*

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
		Make up of weir.		Depth of crest below water surface in							Coefficients of discharge in the various submerged-flow formulas.					
												Formula *b*, observed heads.		Formula *a*, observed heads.		
Name of river.	Name of weir.	Description of parts.	Length of parts, in feet.	Upper pool, feet.	Lower pool, feet.	Observed fall over weir, feet.	Mean surface velocity of approach, in feet per second.	Corresponding velocity head, feet.	Calculated discharge, in cubic feet per second.	Height of main body of weir, feet.	Formula *e*, observed head corrected for velocity of approach.	Corrected for velocity of approach.	Not corrected for velocity of approach.	Corrected for velocity of approach.	Not corrected for velocity of approach.	Approximate section of weir.
			L	h	h_1	$h-h_1=z$	v	$\frac{v^2}{2g}$	Q	p						
Brahmini	Pattia	Main body	665	19.21	18.35	0.86	7.84	0.96	114 000	...	0.881	0.806	1.172	0.241	0.248	
		First step	60	8.21	7.35											
		Second step	9	2.21	1.35											
		Sluices	100	18.10	15.29	2.81			Total 260 000	7.9	0.953	0.940	1.125	0.491	0.502	
	Byturnee	Piers	14	7.68	4.87											
		Main body	912	10.84	8.03											
Byturnee		Total overfall	1 026	.....	.....	.....	8.51	1.13								
		Sluices	100	18.60	15.20	3.40			For main body alone 212 400	7.9	0.979	0.970	1.148	0.511	0.534	
	Burrah	Piers	14	8.18	4.78											
		Main body	412	11.34	7.94											
		Total overfall	526													
		Sluices	389	18.02	17.55	0.47	6.99	0.76	.....	7.5						
Brahmini	Brahmini	Piers	140	8.02	7.55											
		Main body	3 471	10.52	10.05	.....	.....	....	260 000		0.878	0.790	1.230	0.266	0.257	
		Total overfall	4 000	.....	.....	.....	.....	....	307 500		0.876	0.786	1.275	0.245	0.252	
		Sluices	90	30.675	30.15											
Manhanuddy	Kajooree	Breaches	324	25.615	25.09	0.525	11.62	2.10	.....	10.65	.....	.....	.....	.....	.....	
		Main body	3 433	20.115	19.59	.....	.....	....	680 000		0.893	0.764	1.710	0.261	0.274	Same as above.
		Total overfall	3 847	.....	.....	.....	.....	....	787 000		0.891	0.763	1.705	0.355	0.267	
		Shore sluices	190	17.45	15.45											
Manhanuddy	Manhanuddy	Center sluices	460	17.20	15.20	2.00	7.74	0.93	.....	6.00	.....	.....	.....	.....	.....	
		Main body	5 696	11.45	9.45	.....	.....	....	488 000		0.595	0.561	0.679	0.265	0.276	Same as above.
		Total overfall	6 346	.....	.....	.....	.....	....	609 000		0.633	0.597	0.722	0.274	0.284	
		Sluices	255	13.30	12.10											
Manhanuddy	Beropa	Piers	118	4.30	3.10	1.20	6.65	0.68	.....	5.00	.....	.....	.....	.....	.....	
		Main body	1 607	8.30	7.10	.....	.....	....	82 800		0.594	0.570	0.715	0.244	0.255	Same as above.
		Total overfall	1 980	.....	.....	.....	.....	....	112 700		0.648	0.677	0.846	0.260	0.271	

*Compiled and calculated from Paper No. 2171. *Minutes of Proceedings*, Institution of Civil Engineers. Vol. LXXXV, p. 307.

Mr. Nelles. the Humphrey and Abbott Formula, and covers an unusually wide range. Table No. 22, which has been prepared from Mr. Rhind's paper, shows the shape and known dimensions of the dams, the calculated discharge, the observed conditions and deduced coefficients.

In deducing his coefficients, Mr. Rhind used the formula

$$Q = C_1 L \sqrt{2g} \left[h_1 \sqrt{z + 0.01 v^2} + \frac{2}{3} z \sqrt{z + 0.035 v^2} \right] \ldots\ldots\ldots\ldots (e)$$

in which v represents the mean surface velocity of approach. This formula may be reduced to the same form as (b) by neglecting the correction for the velocity of approach.

In addition to the information and results taken from Mr. Rhind's paper, the writer has calculated from the same data and shown in Columns 13 to 16 of Table No. 22, the coefficients of discharge for Formulas (a) and (b) ; first, considering the effect of the velocity of approach, by increasing the observed head an amount equal to $\frac{v^2}{2g}$; second, not considering the effect of the velocity of approach, and using the observed heads in the formulas.

The results in Table No. 22 indicate clearly the necessity for specially considering the effect of the velocity of approach in connection with Formula (b), and show a marked uniformity in the value of the coefficient m in Formula (a), notwithstanding a very wide and inexplicable variation in the values of the coefficient C'.

Van Nostrand's Magazine, Vol. xxxii, page 473, gives the following details of certain observations at the Bazacle Dam in the Garonne River at Toulouse:

$h = 6$ m.;
$h_1 = 5$ m.;
$z = h - h_1 = 1$ m.;
$L = 283$ m.;
$Q = 6500$ cu. m. per second.

Substituting these quantities in Formula (b), we find $C' = 0.915$. Correcting z for a mean velocity of approach of 3.4 m. per second, we find $C' = 0.724$. In the same way, by means of Formula (a), we find $m = 0.353$ and 0.336.

This discussion has been carried to a considerable length in order to illustrate the present state of our practical knowledge on the general question of flow of water over dams, and, aside from the Bazin experiments, much of the matter presented has very little intrinsic value except for this purpose. It is hoped, however, that the illustrations given may attract the attention of skilled investigators to this field, and thus indirectly lead to a more satisfactory solution of the problem of submerged flow.

Some valuable and interesting features of the Bazin experiments

have not been noticed by Mr. Rafter; principal among these are the distribution of pressure and velocity in the nappe. Concerning these features, Bazin says in his summary: Mr. Nelles.

"The pressure under the nappe has been measured in nearly every experiment. For a certain number of them we have determined the interior distribution of both the pressure and velocity. This distribution presents a theoretic interest, but cannot, like the pressure under the nappe, be put to practical use. For a weir of given form, the coefficient m depends principally on two elements, the contraction at the sill and the pressure under the nappe. This pressure is so clearly related to the discharge that we can, by proper observations, deduce from it the variation in the discharge, with a precision not to be obtained by the direct measure of the heads."

GEORGE W. RAFTER, M. Am. Soc. C. E. (by letter).—Mr. Williams is, in general terms, right in his statement that at the beginning of the experiments it was agreed that the reduction of the experiments should be according to Bazin's formula; but so much more time was taken than was expected, that when the writer came to the reductions, and several difficulties developing, due to the experiments going so far beyond the limits of Bazin's formula, it was found necessary to adopt a modified form which would permit of more rapid work. Mr. Rafter.

Mr. Williams has, however, used the Bazin formula rigidly, with the result of showing differences in extreme cases of 3 per cent.

As stated in the paper, Bazin's formula includes the velocity of approach quite as much as any other formula. The reduction of the experiments on the basis of neglecting this element is, for high heads, therefore, an error, and may, with certain others, be briefly discussed. The more especially is this true since it is impossible to work on the ratio $\frac{m}{m'}$.

The following are the more important points, then, in which it appears to the writer that Mr. Williams is in error:

(1) As to the statement that Bazin's formula necessarily provides for the effect of velocity of approach in its coefficient: After determining his two equations, as follows:

$$Q = n\left(1 + \frac{3}{2}\,\alpha\,\frac{u^2}{2gh}\right) l\,h\sqrt{2gh} \quad \ldots\ldots\ldots\ldots \quad (1)$$

and

$$Q = n\left[1 + K\left(\frac{h}{p+h}\right)^2\right] l\,h\sqrt{2gh} \quad \ldots\ldots\ldots \quad (2)$$

Bazin says:

"The first of our two formulas may be found preferable in certain theoretical researches; but the second is obviously more convenient in practice, since it does not contain the velocity of approach u, which itself depends upon the discharge, but only the directly measurable elements, h and p. We, therefore, give it the preference."

Again, on page 258, Bazin says:

"Instead of considering on the other weirs the absolute values of

Mr. Rafter. the coefficient m, we have compared them with the coefficient m' for a free nappe, for the same head on a sharp-crested weir of the same height. This substitution of the ratio $\frac{m}{m'}$, for the absolute values of m eliminates, in a large measure, at least, the influence of velocity of approach, and facilitates greatly the discussion of results."

It appears, therefore, that Bazin did not include the velocity of approach in his formula, and hence its effect is to be allowed for, the same as in the other formulas.

(2) Mr. Williams' view, that the correction for velocity of approach is subtracted, is novel, and will so strike most hydraulicians. It is tolerably clear that he has not understood the language, for certainly the writer has not proceeded in the way that Mr. Williams has assumed. Just how this correction is made will be clear on reference to the correction sheets appended.

(3) It is not believed that the writer has misunderstood Fteley and Stearns. What they actually stated is as follows:

"The head, if measured outside of the angle of pressure, should be taken far enough up-stream from the weir to represent the height of the water surface above the beginning of the surface curvature, *i. e.*, at a distance from the weir equal to $2\frac{1}{2}$ times its height above the bottom of the channel So great a distance from the weir may not be necessary with deep channels; little harm, however, can result from taking the head too far from the weir if the channel is uniform; the amount of error being only the loss of head due to friction."

It would appear, therefore, that the condensed quotation of Fteley and Stearns, namely: "that the only inaccuracy to come from measuring the heads more than 6 ft. back will be due to surface slope," is strictly true.

(4) Mr. Williams' discovery that it is possible to use such a form of weir for a Venturi meter is rather ancient. If he will turn to the *Annales des Ponts et Chaussées*, for 1898, he will find that Bazin discusses this matter very extensively, and concludes in the following words:

"The measure of pressure can then be utilized for registering varying flows. An instrument founded upon an application of this principle has been used for a number of years in America for measuring the flow of large water conduits. It is understood, from a well-known experiment of Venturi, that if we restrict for a short distance the diameter of a conduit, it will produce, in the part restricted, a diminution of pressure corresponding to the increase in velocity of the fluid, etc. * * * The apparatus designed by Clemens Herschel is founded on this principle. The same principle is evidently applicable to weirs where one can realize directly the registration of the discharge by employing the process of Mr. Hegly for transforming the indication of height on a weir into indications of flow."

(5) Mr. Williams details an experiment as follows:

"At the time that these heads were read a tape was tacked upon the wall of the canal vertically at the crest, so that the top of the sheet could be read thereon at the same time that the pressure in the piezome-

ter along the crest of the weir was read. With a head up stream of 97 cm. the tape read 70 cm. and the piezometer at the crest read 38 cm." Mr. Rafter.

Mr. Williams then adds: "There is the effect of velocity upon the head." Exactly what Mr. Williams means by the effect of velocity upon the head, the writer does not know, but the obvious interpretation of the experiment is that there is a suction at the crest, as pointed out by Bazin some time ago, and that the piezometer reading 38 cm. merely recorded such suction.

(6) After somewhat careful study of the work of Francis, of Fteley and Stearns, and of Bazin, the writer is obliged to differ from Mr. Williams in regard to the accuracy with which the respective experimenters did their work. He has no doubt that Bazin's work was quite as accurate as that of the others. Mr. Williams, moreover, raises a question of patriotism which, it seems to the writer, would be better left unsaid.

(7) We come now to the question of the proper location of the piezometers. As detailed in the paper, two sets of piezometers were used. In one case a 1-in. galvanized iron pipe, with holes $\frac{1}{4}$ in. in diameter and spaced 6 ins. apart, was laid across the channel about 8 ins. above the bottom, with the holes therein opening downward. Connections with these pipes were made by $\frac{3}{4}$-in. pipes passing through the bulkhead to a point below the weir, where the gauges could easily be connected by rubber hose. In order to check the accuracy of the piezometric readings, at the conclusion of Experiment No. 17, a fourth piezometer pipe was set in the bottom of the flume above the lower bulkhead, and about 6 ins. up stream from the upper piezometer. This pipe was set with $\frac{1}{4}$-in. holes directly on top, and with the top of the pipe flush with the bottom of the flume. As stated, considerable differences were noted in simultaneous readings of these two piezometers.

Mr. Williams describes the piezometer which he finally adopted as raised 6 ft. from the bottom, and he apparently asks that measurements taken thereon shall be considered more accurate than when taken on a piezometer flush with the bottom. As a reason for adopting this type of piezometer, he assigns that Mr. FitzGerald used it in his experiments on the flow of water in a 48-in. pipe (except that Mr. FitzGerald's piezometer was on the bottom), although it seems clear enough that the difference of condition would indicate no comparison between Mr. FitzGerald's method and the work at Cornell. It is perfectly true that a memorandum sufficient to locate a correction curve was received from Mr. Williams and rejected, for the reason that his views on piezometers, particularly that a piezometer 6 ft. from the bottom was preferable to one flush with the bottom, were not considered especially sound. For this, the writer is fully responsible. It is not intended to state, however, that the flush piezometer was absolutely the best that

Mr. Rafter. could be applied, but it seems to be clear enough that it was better than one 6 ft. up, where the current necessarily must have been considerable.

It seems difficult to understand why there should be so much uncertainty about piezometers, the more especially as Mr. Hiram F. Mills has pointed out in his "Experiments upon Piezometers used in Hydraulic Investigations"* the proper method of their use. In his conclusions Mr. Mills states:

"This result indicates, with a nearness of approximation unusual in hydraulic investigations, that with the plane of the orifice accurately in the plane of the side of the conduit, the piezometer will indicate the true height of the surface of the stream."

Without going into detail, it may be remarked that the writer believes that, in order to obtain accurate results, all Mr. Williams had to do was to place the plane of the orifice accurately in the plane of the side of the conduit. Evidently, from the experiments detailed, he finally reached this result; thus experimentally realizing a method which has been the common property of engineers for many years.

It is clear, also, that he has considerable doubt as to the accuracy of his method of measurement, because he finally states that "it cannot be affirmed that the new piezometer is a correct one to use with Bazin's formula."

(8) Mr. Williams' argument as to the method of reduction having been shown to be wrong, it follows that his tables have no special significance, and his expression of opinion, that the results do not come nearer to accuracy than 6%, even though on his own unfavorable assumption he is unable to find more than 3% error, is, it seems to the writer, going somewhat farther than is necessary.

(9) Mr. Williams criticises the brevity with which the results are presented, his view being that the experiments should be presented with as little reduction as possible, "so that in the future the investigator may determine for himself, in the light of such new knowledge as he may then have, just what reliability is to be put upon the observations, and what lessons are to be drawn from them."

This view is based upon the notion that all the work done by engineers is highly scientific. As a matter of fact, the engineer is not specially a man of science, but, nevertheless, he should obtain accurate results. The method of writing in such detail as Mr. Williams proposes has always seemed to the writer somewhat sophomoric. He has the detail, however, and takes pleasure in presenting it, although the paper has already grown to undue length.

(10) In Figs. 21 and 22, Mr. Williams presents a series of sketches of the curves formed by the water as it flows over the various weirs. Inasmuch as Bazin has presented similar sketches, but so much more accurate than his, it is difficult to see the utility of this presentation.

* *Proceedings*, American Academy of Arts and Sciences, 1878.

(11) Mr. Williams omits Series Nos. 1, 2 and 6, and also considers Series No. 5 as quite possibly inaccurate. The writer considered Series Nos. 1, 2, 3 and 4 as less accurate than the others. In regard to these, it may be remarked that the piezometric readings were not as accurate as might be desired. Fortunately, the actual elevations have been kept, and have been used in the reductions. The actual elevations, however, were not measured closer than about $\frac{1}{100}$ ft., and within that limit there may be some variation. Otherwise, these four experiments are as reliable as the others. Mr. Rafter.

(12) Mr. Williams states that:

"One of the most important facts brought out in the past year's investigations in the Cornell Hydraulic Laboratory has been the formation of a vacuum more or less perfect behind the falling sheet when air is not freely admitted."

Why Mr. Williams should claim this as an original discovery is difficult to determine, the more especially since Bazin has discussed it very extensively.

(13) Mr. Williams' erroneous view as to the proper method of computation by Bazin's formula has a number of important considerations connected therewith. For instance, he is apparently under the impression that the influence of the height of the weir is a matter of so much importance as to require computing to the second or third decimal place. The following demonstration, for which the writer is indebted to Alfred Noble, M. Am. Soc. C. E., may serve to set him right on this question: Taking Bazin's formula—

$$m = n\left[1 + 0.55\left(\frac{h}{h+p}\right)^2\right]$$

we can deduce

$$\frac{n}{m} = \frac{1}{1 + 0.55\left(\frac{h}{h+p}\right)^2}$$

We can then calculate values of $\frac{n}{m}$ for assumed values of $\left(\frac{h}{h+p}\right)$, obtaining the following table:

$\left(\frac{h}{h+p}\right)$	$\left(\frac{n}{m}\right)$
0.1	0.995
0.2	0.978
0.3	0.953
0.4	0.919
0.5	0.879
0.6	0.835
0.7	0.788
0.8	0.740
0.9	0.692
1.0	0.645

Mr. Rafter. This formula is empirical, is not applicable to extreme cases, and is obviously absurd for values of $\left(\frac{h}{h+p}\right)$ approaching unity.

(14) The Croton gaugings have been referred to, and Mr. Williams expresses the opinion that they are considerably more accurate than those now under discussion. On this point the writer desires to state that without making a very thorough study of the Croton gaugings, he has computed the values of m and plotted coefficient curves similar to those on pages 267–284. These curves are not especially reassuring as to the minute accuracy of the results. Some of them are undoubtedly in error.

Again, consulting the detailed tabulation, as published in Mr. Freeman's Report on the New York Water Supply, pages 137–141, it is learned that, out of 139 experiments, 44 were omitted because of uncertainty of some sort in the detail. This is nearly 32 per cent. As indicating a probable reason why these experiments are somewhat unreliable, it may be noted that they were all made with a longitudinal piezometer, raised 6 ft. from the bottom; which position the writer believes is fundamentally wrong. Mr. Freeman will understand, it is hoped, that the slight criticism of the Croton experiments made here is not in any way directed toward him. It is directed alone to Mr. Williams' statement that the Croton experiments are considerably less than 6% in error.

(15) Mr. Williams has referred to a Francis piezometer. This also seems to be an error, as the writer does not now remember that Francis used anything that could with propriety be called a piezometer. In Francis' original experiments, the heads were measured in "still boxes" which were set in the hollow quoins of the canal locks. These "still boxes" had a hole in the bottom, and were set in practically still water, the level of the water being determined by a hook gauge. Later, other experimenters set the "still boxes" at any convenient place and connected them with the water above the weir by means of pipes.

The piezometer described on page 322 is the Fteley and Stearns piezometer.

The writer very readily admits that the several formulas are not strictly comparable, and has therefore revised the Bazin experiments to conform therewith. This revision involves the dropping of the C from $C = m\sqrt{2g}$, retaining only $m\sqrt{2g}$. In the case of the Cornell experiments, however, owing to the method of reduction being different, the expression $C = M\sqrt{2g}$ is perfectly true, and has accordingly been retained. Practically, the difference is so small as to be, in the great majority of cases, safely neglected; but, in order to keep the distinction between the theoretically true and practically true distinct, the change is made. Still, for all ordinary cases, the Bazin coefficients may be used in the expression $C = m\sqrt{2g}$, thus very greatly extending the range of weir formulas.

Mr. Rafter.

In his pamphlet on "Measuring Water," Clemens Herschel, M. Am. Soc. C. E., has discussed nearly every phase of the question, and defined with great clearness the more important elements of the problem. This pamphlet may, in the writer's opinion, be profitably read by anybody having occasion to use the weir.

(16) Mention has been made, at the beginning, of special difficulties of reduction. These relate more especially to the mean values of a and K. For instance, within the limits of Bazin's experiments, a ranges in value from 2.43 to 1.40, and K from 0.77 to 0.50. These ranges in value occur for weirs from 2.46 ft. (0.75 m.) in height to 0.79 ft. (0.24 m.) in height. The standard weir of the Cornell experiments was 13 ft. in height and the experimental weir about 5 ft. The values used are means derived from a considerable number of cases. These are given in tables.*

These tables show:

(1) That the values of a decrease with decrease of height of weir, and that, for a given height, they first increase with the head, up to a maximum, and then again decrease.

(2) That the values of K, which vary less than those of a with the height of the weir, become more nearly equal as the head increases.

There seems to be a general misconception as to Bazin's use of the correction for velocity of approach. In regard to this element, Bazin states:

"When it is required to take into account the effect of the velocity of approach to the weir, it is usual, in the very simple formula,

$$Q = m\, l\, h\, \sqrt{2\, g\, h} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

to substitute $\left(h + a\,\frac{u^2}{2\,g}\right)$ for h, where u is the mean velocity in the channel leading to the weir and a a coefficient not very accurately determined, but generally taken at 1.5. The equation thus becomes:

$$Q = n\, l \left(h + a\,\frac{u^2}{2\,g}\right) \sqrt{2\,g\left(h + a\,\frac{u^2}{2\,g}\right)}$$

$$= n\, l\, h \sqrt{2\,g\,h}\left(1 + a\,\frac{u^2}{2\,g\,h}\right)^{\frac{3}{2}}" \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

After a series of transformations, this reduces to

$$Q = n\left(1 + \frac{3}{2}\, a\,\frac{u^2}{2\,g\,h}\right) l\, h\, \sqrt{2\,g\,h} \quad \ldots\ldots\ldots\ldots\ldots (3)$$

an expression which may be written in nearly equivalent form,

$$Q = n\left[1 + K\left(\frac{h}{h+p}\right)^2\right] l\, h\, \sqrt{2\,g\,h} \quad \ldots\ldots\ldots\ldots\ldots (4)$$

which does not contain the velocity of approach, u, but only the directly measurable elements, h and p.

* See *Annales des Ponts et Chaussées* for 1888; and *Proceedings*, Engineers' Club of Philadelphia, Vol. VII, No. 5. January, 1890.

Mr. Rafter. We, therefore, Bazin states, adopt definitely for m the formula which gives, for the discharge Q,

$$Q = n\left[1 + 0.55\left(\frac{h}{p+h}\right)^2\right] l\, h\, \sqrt{2gh},$$

and for value of M,

$$M = n\left[1 + 0.55\left(\frac{H}{p+H}\right)^2\right].$$

It is hoped it will be understood that this is a very brief exposition of these formulas. Several pages are condensed into one. So far as the writer is concerned, he had no intention of writing a treatise on the weir.*

The writer is aware that a slight uncertainty attaches to the position of the piezometer, as noted by Mr. Parmley, nor would he be satisfied with readings to 2 mm. in measuring low heads. It seems, however, that Mr. Parmley's criticism is, on the whole, pointless, precisely because the heads are large. Indeed, his demonstration is pertinent on this point.

As to the various criticisms relative to the form of the coefficient curves, the writer wishes to state that, on the whole, they seem to indicate a lack of familiarity with the subject, which, indeed, is not to be wondered at, considering how thoroughly engineers have been wedded to the view that Francis' formula was all there was to the weir. Apparently, the insignificant causes which may produce a very material variation in the form of the curve are not appreciated. Moreover, a very considerable variation in the form of the curve may be produced by varying the value of the coefficient only very slightly, as fairly illustrated by comparing the two curves of Experiment No. 6. The writer cannot but think, therefore, that Mr. Wisner has been somewhat too hasty in his conclusion that some of the curves are wrong. As the matter stands, there is not, in the writer's opinion, any basis for the judgment that the curve for Experiment No. 19 is wrong. As to the observations made since these experiments were completed, and which show material error in these observations, it may be merely pointed out that as yet they have not materialized.

A portion of Mr. Parmley's discussion is conflicting. To begin with, he states (page 349):

"If the writer understands Bazin's formula, no correction for velocity of approach is necessary, since that is taken account of already in the formula and the coefficients given in his table."

On page 350, he states:

"One of the first things noticed was the fact that in correcting for velocity of approach, Bazin followed practically the method of Francis, but with a coefficient which gives a larger correction."

* In order to obtain the whole matter conveniently, the reader is referred to the translation by Marichal and Trautwine, *Proceedings*, Engineers' Club of Philadelphia Vol. VII, No. 5, January, 1890.

Mr. Rafter.

A word as to the standard weir, 16 ft. in length: Mr. Kuichling has expressed a doubt as to its accuracy, and cites Hamilton Smith's remark, "such speculations, though possibly ingenious and plausible, when tested with facts, generally prove to be very wide of the truth." Mr. Kuichling might, with great appropriateness, have quoted the preceding sentence. As he failed to do so, probably from mere inadvertence, the writer will quote it: "It is hardly worth while to pursue this subject still further into spaces where we have absolutely lost the guidance of experimental light." Does Mr. Kuichling mean to affirm that we have "absolutely lost" the guidance of experimental light in these experiments? As to his assumed error in computation, on looking over the detail of the experiments he may conclude that possibly he is in error in hastily assuming error in computation.

Probably the most appreciative discussion is that of Mr. Nelles, who evidently has had occasion to study the subject broadly. Mr. Horton's discussion is also one from a man who has had to deal with large weirs. The writer congratulates himself on having called Mr. Nelles' attention to Bazin's work, and hopes that the study so well begun may be continued to a successful issue. Mr. Nelles thinks, however, that there may be a question either as to the accuracy of these experiments or their applicability to the flow over structures differing in shape and dimensions from the experimental forms. On this point the writer again calls attention to what seems to him ought to be understood by everybody, namely, that very slight change in the form of the weir often produces relatively important changes in the form of the coefficient curve. Reasoning from one coefficient curve to another is, therefore, without the slightest significance. This point is again referred to because of a general misapprehension by those who have contributed to this discussion.

Mr. Kuichling gives an account of the foundation of Bazin's formula, although the writer cannot but think it would have been much more acceptable if he had included some account of how Bazin corrects for velocity of approach. Lacking such, it indicates rather casual reading of Bazin's paper.

In the writer's opinion, Mr. Williams is wrong in his view that Bazin intended to apply his formula for height of weir rigorously. In Table No. 2,* for a height of 6.56 ft. above the bottom of the channel, the coefficient m does not vary by so much as one decimal place between the limits of heads of 0.30 and 0.60 m. The preceding discussion may serve to indicate why this correction is a somewhat general one, and not intended to be rigorously applied beyond the limit.

With reference to the method used in calibrating the standard weir, the writer is unable to see any difficulty about it, but inasmuch as there is clearly some misapprehension he will endeavor to clear away

* See, also, *Annales des Ponts et Chaussées*, for 1888, p. 446.

Mr. Rafter. the doubt which has arisen in the minds of some as to just what was done.

The reasons why the flow in the experimental canal was not checked have already been brought up in the discussion. The writer's contention is that this was not specially necessary; that all that had to be done was to absolutely duplicate Bazin's experiments; and, up to a height of 2 ft. on the 16 ft. weir, at any rate, the error could not have been more than 1 per cent. As to the length of the weirs affecting the results, as suggested by Mr. Kuichling, Bazin's decisive experiments on his 1-m. and 2-m. weirs ought to be conclusive. He states distinctly that he was unable to note any difference in the results. It follows, therefore, that if there was no difference up to 2 m., there is no reason for supposing any difference up to 4 or 5 m. Owing to the fact that the arrangements were not absolutely duplicated, the writer thinks it quite probable that the error may range from 1 to 2 per cent. Above 2 ft., there is, of course, more question. Indeed, the writer realized that Bazin's formula, from its peculiar composition, as already pointed out in the discussion, might not be entirely reliable beyond the limit of his table.*

Column 3, of Table No. 23, gives values of n.

TABLE No. 23.

Head, in meters. h.	Head, in feet. h	n	$\frac{h}{p+h}$	$\left(\frac{h}{p+h}\right)^2$	$1+0.55\left(\frac{h}{p+h}\right)^2$	M.
(1)	(2)	(3)	(4)	(5)	(6)	(7)
0.10.....	0.3281	0.4322	0.0244	0.000595	1.000327	0.4323
0.20.....	0.6252	0.4215	0.0476	0.002266	1.001246	0.4222
0.30.....	0.9843	0.4147	0.0698	0.004872	1.002680	0.4185
0.40.....	1.3124	0.4144	0.0909	0.008263	1.004544	0.4163
0.50.....	1.6405	0.4118	0.1111	0.012343	1.006789	0.4146
0.60.....	1.9686	0.4092	0.1305	0.01703	1.009366	0.4133

A discharge curve was computed for the 16-ft. standard weir, using the values of n given in Table No. 23, and from this curve the discharge over the standard weir was taken out for such observed heads in Experiments Nos. 20 and 21, as did not exceed 1.969 ft. or 0.6 m., which is the limit of the table, and it was found that the flow over the 16-ft. standard weir, under a head of 1.969 ft. produced a head of 3.57 ft. on the 6.58-ft. sharp-crested experimental weir. We have thus obtained a series of heads on, and flows over, the 6.58-ft. weir, up to 3.57 ft. head. From this point, Bazin's formula was no longer used.

The velocity of approach in the channel leading to the experimental weir was next determined directly, which was also done in all experiments

* *Annales des Ponts et Chaussées*, for 1888, p. 446 (also given as Table No. 2 of this paper).

on irregular sections, by dividing the volume of flow already known, by the actual cross-sectional area of the channel of approach. The corresponding velocity heads, so obtained, were added to the observed heads on the experimental weir, and the discharge coefficient computed by the formula, Mr. Rafter.

$$M = \frac{Q}{L\,H\sqrt{2\,g\,H}}$$

We have obtained by the preceding operations, a series of discharge coefficients for the sharp-crested weir, 6.58 ft. in length, for heads up to 3.57 ft. A discharge curve was then computed therefrom for the 16-ft. sharp-crested weir for heads up to 3 ft. on that weir, as given in Fig. 4. That is to say, the operation was, in effect, reversed. We had first obtained, with heads of 1.969 ft. on the 16-ft. standard weir, a corresponding head of 3.57 ft. on the 6.58-ft. experimental weir, and their coefficients are considered accurate to within about 1 per cent. A discharge curve was then plotted from which readings from the 6.58-ft. experimental weir were applied, back to about 3 ft. on the 16-ft. standard weir, thus calibrating that weir up to 3 ft., and consequently making it possible to obtain readings up to 5.89 ft. on the 6.58-ft. experimental weir. This explanation, it is hoped, will clear up any uncertainties as to just why the 6.58-ft. weir was used.

In obtaining the coefficients used in plotting the foregoing curve, the velocity head was added to the observed head, and hence the velocity head at the standard weir should be, and has been, added to the observed head in reducing the other experiments; but as the velocity head is a function of the discharge, it must, in this case, be determined by successive approximations. This was done in a manner similar to that already explained, and gives a continuous discharge curve for heads within the limit of the experiments, or say from about 0.5 ft. to 5 ft.

These coefficients are in a form which makes them applicable, without modification, to a weir of any height whatever. The effect of the height of the weir on the velocity of approach is therefore included in the velocity head. Mr. Williams is mistaken in the statement that they give a discharge curve for the 16-ft. standard weir agreeing with that by Bazin's formula, while that for the lower 6.58-ft. weir does not do so. It remains, therefore, only to point out that we know nothing of the flow over the experimental weirs except as determined from known flows over the standard weir. It is a mere waste of time to attempt to apply formulas for flow to the experimental weirs, as it is, also, to attempt to reason from known forms which have been experimented upon, to unknown forms. However unsatisfactory it may appear to those who are looking for simplicity, the present tendency in hydraulics is to great complexity.

The writer is aware that the point may with propriety be discussed as to whether or not absolutely the best correction for velocity of

Mr. Rafter. approach was applied in this case. All that he can state is that the pressure for immediate results was very great, and that for the time available the best was done.

For reasons given on page 301, the writer did not feel justified in fixing the percentage of probable error. Recently, however, a careful study has been made of all the data and circumstances attending the making of the experiments, and the writer, in consequence, is of the opinion that at the limit of the highest heads the error may possibly be somewhat over 2 per cent. Even though it were 3%, as Mr. Williams claims, he should not feel specially dissatisfied, because the difficulties in making an absolutely correct measurement are enormous. In any case, the writer hopes that a possible error of 3% in the determinations will not be fatal to the experiments. Up to 3.57 ft. on the experimental 6.58-ft. weir, it is probable that the error does not much exceed 1 per cent. If, therefore, Mr. Williams will kindly recompute, with proper allowance for velocity of approach at both weirs, it is believed that he will find the error to be smaller than he has imagined.

Another point in which this work differs from Bazin's may be mentioned: Bazin worked on weirs of substantially the same length, and could thus avail himself of the relation $\frac{m}{m'}$ for neutralizing the effect of velocity of approach. In the present case the standard weir was 16 ft. in length and the experimental weir was 6.58 ft. in length. It was impossible, therefore, to proceed in the same manner as Bazin.

A word as to the future of weir measurements: At the present time there is certainly a good deal that is unsatisfactory about them. Bazin's formula is undoubtedly the best that has been thus far devised, but is it, after all, the best that can be devised? Bazin has evidently realized the unsatisfactory nature of weir measurements as now conducted, for he expresses himself strongly that preferably such measurements should be made by means of pressure observations in the nappe.* From such observations it is possible there would come a decrease in the number of coefficients. The writer regrets that he is not able to offer any suggestions as to the best method of proceeding with a series of nappe observations. Mr. Williams has here an opportunity to employ himself usefully for several years.

Time will not be taken to point out the many practical relations embodied in the new views as to weirs. One deduction may, however, be pointed out, namely, the effect as to gaugings of the run-off of rivers. Wherever there were convenient dams, thus far, engineers have used Francis' formula almost exclusively for such work, although it is clear enough now that, without understanding the great variation on weirs of different forms, this formula, with its fixed coefficient, is in many cases in no way applicable, the variations being from 20 to

* See, also, an article by Clemens Herschel, M. Am. Soc. C. E., in *Engineering News*, November 10th, 1898.

40 per cent. Recently, there has been considerable discussion as to the effect of forests on run-off, but inasmuch as such effect is considerably within the limit of variation in Francis' formula, it is obvious that gaugings based thereon are too uncertain to furnish any light whatever on this subject. As far as the writer can now see, it will be necessary to recompute most of the gaugings, before this question can be settled. Mr. Rafter.

Possibly due to a slight confusion in the use of H and h, there has been some misapprehension as to just how the Cornell coefficients are to be used. At the risk of being somewhat elementary, the writer will give an illustration: In the formula, $Q = C\,L\,H^{\frac{3}{2}}$, the coefficient, C, varies according to the head instead of remaining constantly equal to 3.33, as in Francis' expression. Thus, if it is required to compute the flow over a dam of the form given by the curve on page 274 (Cornell Experiment, No. 9) for $H = 0.5$ ft., $C = 3.300$. When $H = 1$ ft., C will $= 3.570$, and when $H = 1.5$ ft., C will $= 3.595$, and so on. The values of H, it may be repeated, are corrected for velocity of approach, and are to be used without further correction of any kind.

The writer confesses to a feeling of disappointment at the results of the discussion. He had hoped that the numerous practical questions which are raised would receive extensive consideration, but, unfortunately, the discussion has been almost entirely about relatively unimportant theoretical questions. This, the writer, although in no way responsible for, very much regrets.

Enough of the tabulations of the detailed results are herewith presented to show clearly the method of reduction. It is considered that they are sufficiently self-explanatory to be readily understood by anybody. They were not given in the original paper because of its great length.

TABLE No. 24.—Experiment No. 1.—Bazin's Series No. 130.

Computed from Heads Reduced to Gauge Board at Experimental Weir.

Period.	Observed head, upper piezometer, standard weir.	Correction for middle piezometer.	Corrected head, in centimeters.	Corrected head, in feet.	Correction for velocity of approach.	Final head on standard weir, in feet.	Flow over standard weir, in cubic feet per second.	Flow over experimental weir. Corrected for leakage.	Flow over experimental weir, per foot of crest.	Observed head, experimental weir.	Mean correction, to reduce to gauge-board reading.	Corrected head, in feet.	Final corrected head on experimental weir.	Q per foot.	$C = M\sqrt{2g}$.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
1...	85.59	2.94	88.53	2.905	0.016	2.920	261.7	261.21	39.73	5.494	0.522	4.972	4.972	39.73	3.584
2...	85.12	2.92	88.04	2.889	0.016	2.904	260.0	259.51	39.44	5.394	0.522	4.872	4.872	39.44	3.668
3...	85.00	2.91	87.91	2.884	0.016	2.900	259.2	258.71	39.31	5.375	0.522	4.853	4.853	39.31	3.940
4...	73.19	2.32	75.51	2.477	0.011	2.488	207.5	207.05	31.47	4.492	0.354	4.138	4.138	31.47	3.739
5...	59.41	1.68	61.09	2.004	0.006	2.010	150.1	149.70	22.81	3.573	0.205	3.368	3.368	22.81	3.690
6...	31.37	0.59	31.96	1.049	0.001	1.049	57.7	57.40	8.71	1.755	0.030	1.725	1.725	8.71	3.844
7...	21.49	0.29	21.78	0.715	0.0003	0.715	32.4	32.13	4.88	1.190	.....	1.190	1.190	4.88	3.759

Mr. Rafter.

TABLE No. 25.—Experiment No. 5. May 27th, 1899. A. M.

Period.	Observed head on middle piezometer, standard weir, in feet.	Correction for velocity of approach, in feet.	Corrected head, in feet.	Flow over standard weir, in cubic feet.	Flow over experimental weir, in cubic feet. Corrected for leakage.	Observed head on experimental weir, in centimeters.	Correction for flush piezometer, in centimeters.	Corrected head, in centimeters.	Corrected head, in feet.	$p + h$.	r_o.	$\frac{V_o^2}{2g}$	Final corrected head, in feet.	Q per foot.	$C = M\sqrt{2g}$.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
1..	2.956	0.0169	2.973	268.5	268.0	138.11	9.25	147.36	4.834	9.730	4.15	0.251	5.085	40.73	3.552
2..	2.486	0.0108	2.497	208.1	207.65	117.35	5.88	123.23	4.043	8.939	3.51	0.191	4.234	31.56	3.623
3..	1.951	0.0059	1.956	144.0	143.65	95.72	3.40	99.12	3.252	8.148	2.76	0.119	3.371	21.83	3.527
4..	1.471	0.0023	1.473	95.0	94.66	74.01	1.45	75.49	2.477	7.373	1.95	0.062	2.539	14.38	3.554
5..	1.014	0.0008	1.014	54.0	53.70	50.02	0.73	50.75	1.665	6.561	1.24	0.024	1.689	8.16	3.785
Experiment No. 6. May 27th, 1899. P. M.															
1..	.....	.....	2.806	247.0	246.5	129.61	7.95	137.56	4.513	9.622	3.86	0.232	4.745	37.45	3.623
2..	.....	.....	2.538	213.0	212.55	116.33	6.15	122.48	4.018	9.127	3.51	0.192	4.210	32.30	3.739
3..	.....	.....	2.048	154.3	153.90	94.77	3.60	98.37	3.228	8.337	2.71	0.114	3.342	23.38	3.827
4..	.....	.....	1.494	96.5	96.15	73.14	1.62	74.76	2.453	7.562	1.92	0.058	2.511	14.61	3.672
5..	.....	.....	1.062	59.0	58.70	50.21	0.80	51.01	1.674	6.783	1.17	0.021	1.695	8.92	4.042
Experiment No. 16. June 5th, 1899.															
1..	2.4657	0.0106	2.4763	206.00	205.55	120.47	5.70	126.17	4.140	8.708	3.599	0.199	4.339	31.238	3.456
2..	1.9647	0.0057	1.9704	145.40	145.00	98.97	3.34	102.31	3.357	7.925	2.789	0.125	3.406	22.036	3.406
3..	1.4696	0.0026	1.4722	94.50	94.15	76.19	1.57	77.76	2.551	7.119	2.015	0.063	2.614	14.308	3.386
4..	0.9738	0.0007	0.9745	51.80	51.50	51.48	0.67	52.15	1.711	6.279	1.248	0.024	1.735	7.826	3.424
Experiment No. 17. June 5th, 1899.															
1..	2.4644	0.0106	2.475	205.50	205.05	121.04	5.70	126.74	4.1583	8.726	3.57	0.178	4.336	31.16	3.491
2..	1.9660	0.0057	1.972	146.00	145.60	97.76	3.33	101.09	3.3168	7.885	2.81	0.123	3.440	22.13	3.468
3..	1.4774	0.0026	1.480	95.70	95.35	75.55	1.63	78.18	2.5651	7.133	2.03	0.062	2.627	14.49	3.403
4..	0.9692	0.0007	0.970	51.40	51.00	51.02	0.70	51.72	1.6969	6.265	1.24	0.024	1.721	7.75	3.433
5..	0.5932	.....	0.593	24.50	24.25	32.01	0.30	32.34	1.0612	5.629	0.65	0.006	1.067	3.68	3.339
Experiment No. 18. June 6th, 1899.															
1..	0.7668	0.0004	0.767	36.00	35.73	38.52	Used flush piezometer.		1.2638	5.91	0.92	0.013	1.277	5.43	3.763
2..	2.9440	0.0167	2.961	267.1	266.60	149.35			4.9001	9.55	4.24	0.280	5.180	40.52	3.437
3..	2.4486	0.0104	2.459	203.5	203.05	125.69			4.1239	8.77	3.52	0.193	4 317	30.86	3.440
4..	1.9469	0.0056	1.952	143.5	143.10	100.83			3.3782	7.96	2.73	0.116	3.424	21.75	3.433
5..	1.4479	0.0025	1.450	92.5	92.15	75.48			2.4765	7.13	1.97	0.061	2.537	14.06	3.479
Experiment No. 19. June 7th, 1899.															
1..	0.4632	.....	0.463	17.10	16.85	26.95	Used flush piezometer.		0.8842	6.162	0.41	0.003	0.887	2.56	3.063
2..	0.4731	.....	0.474	17.60	17.35	27.04			0.8872	6.165	0.42	0.003	0.890	2.64	3.144
3..	0.9492	0.0007	0.950	49.50	49.21	51.36			1.6851	6.963	1.08	0.018	1.703	7.48	3.366
4..	1.0014	0.0008	1.002	54.00	53.70	53.42			1.7527	7.031	1.16	0.021	1.774	8.16	3.454
5..	1.4725	0.0026	1.475	95.00	94.65	77.02			2.5270	7.805	1.84	0.053	2.580	14.38	3.470
6..	1.9092	0.0053	1.915	139.50	139.11	97.84			3.2101	8.488	2.49	0.097	3.307	21.14	3.515
7..	2.3961	0.0099	2.406	197.00	197.56	119.34			3.9155	9.194	3.25	0.165	4.080	30.03	3.644
8..	2.9112	0.0162	2.927	262.90	262.44	142.13			4.6633	9.947	4.01	0.250	4.913	39.88	3.662

TABLE No. 26.—Reductions for Experiments Nos. 20 and 21, where the Head on Standard Weir was less than 2 ft. Mr. Rafter.

Experiment.	Period.	Head on standard weir from middle piezometer, in feet.	Total flow over standard weir by Bazin's formula, in cubic feet.	Flow over experimental weir, corrected for leakage.	Observed head on experimental weir, in centimeters.	Observed head on experimental weir, in feet.	$p + h$.	V_o.	$\frac{V_o^2}{2g}$	Final corrected head on experimental weir.	Flow over experimental weir per lineal foot.	$C = \frac{}{\sqrt{2g}}$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
20..	1	0.99	53.0	52 7	55.21	1.8114	7.07	1.11	0.016	1.827	8.07	3.272
20..	4	1.96	145.2	144.8	105.92	3.4752	8.73	2.44	0.093	3.568	22.18	3.291
20..	5	1.465	94.5	94.15	80.56	2.6432	7.90	1.76	0.048	2.691	14.42	3.267
21..	1	0.598	25.2	24.94	33.12	1.087	6.34	0.59	0.006	1.093	3.82	3.343
21..	2	0.795	38.5	38.22	44.00	1.444	6.70	0.80	0.010	1.454	5.85	3.337
21..	3	1.097	54.5	54.20	55.13	1.809	7.07	1.11	0.016	1.825	8.30	3.290
21..	4	1.25	74.8	74.48	68.17	2.237	7.49	1.97	0.033	2.270	11.41	3.335
21..	5	0.896	45.8	45.51	49.72	1.631	6.89	0.98	0.015	1.646	6.97	3.301
21..	6	0.710	32.5	32.23	39.42	1.293	6.55	0.74	0.009	1.302	4.94	3.322

TABLE No. 27.—Computations for Periods (2) and (3), Experiment No. 20, Taking Discharge over Standard Weir from Curve computed from Results of Experiments previously reduced.

Period.	Observed head on middle piezometer.	Correction for velocity of approach.	Corrected head on standard weir.	Total flow over standard weir, in cubic feet per second.	Flow over experimental weir.	Observed head on experimental weir.	$\frac{V_o^2}{2g}$	Corrected head on experimental weir.	Flow, per foot of length, over experimental weir.	$C = M\sqrt{2g}$.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
2.....	2.693	0.013	2.706	233.28	232.81	4.672	0.198	4.875	35.68	3.315
3.....	2.441	0.011	2.452	201.76	201.32	4.274	0.160	4.434	30.83	3.302

Mr. Rafter.

TABLE No. 28.—Cornell Experiments. Summary of Elevations.

Experiment.	Date.	Standard Weir.				Experimental Weir.				
		Mean elevation of crest.	Elevation gauge zero.	Difference, in centimeters.	Piezometer used in reduction of experiments.	Mean elevation of crest.	Elevation gauge zero.	Difference, in centimeters.	p = height of crest above channel bottom.	Piezometer used in reduction of experiments.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
1	May 20	95.256	97.920	81.38	Up-stream P.	86.640	86.640	.....	4.700	Board.
2	" 24	95.258	97.909	80.80	" P.	86.827	86.827	.....	4.887	"
3	" 26	95.258	94.262	30.36	" P.	86.847	86.827	.....	4.907	"
4	" 26	95.258	94.262	30.39	" P.	86.847	86.827	.....	4.907	"
5	" 27	95.258	94.261	30.39	Middle P.	86.836	87.342	15.42	4.896	Up-stream P.*
6	" 27	95.258	94.261	30.39	" P.	87.049	87.342	8.93	5.109	" P.
7	" 29	95.258	94.261	30.39	" P.	86.835	87.342	15.45	4.895	" P.
8	" 30	95.258	94.259	30.45	" P.	86.835	87.349	15.67	4.895	" P.
9	" 31	95.258	94.259	30.45	" P.	86.881	87.349	14.26	4.941	" P.
10	June 1	95.258	94.259	30.45	" P.	86.510	87.349	25.57	4.570	" P.
11	" 1	95.258	94.259	30.45	" P.	86.510	87.349	25.57	4.570	" P.
12	" 2	95.258	94.259	30.45	" P.	86.504	87.349	25.76	4.564	" P.
13	" 2	95.258	94.259	30.45	" P.	86.504	87.349	25.76	4.564	" P.
14	" 3	95.258	94.259	30.45	" P.	86.469	87.349	26.82	4.529	" P.
15	" 3	95.258	94.259	30.45	" P.	86.469	87.349	26.82	4.530	" P.
16	" 5	95.258	94.259	30.45	" P.	86.508	87.349	25.63	4.568	" P.
17	" 5	95.258	94.259	30.45	" P.	86.508	87.349	25.63	4.568	" P.
18	" 6	95.258	94.259	30.45	" P.	86.590	87.349	23.13	4.650	Flush P.
19	" 7	95.258	94.259	30.45	" P.	87.218	87.349	3.99	5.278	" P.
20	" 10	95.258	94.259	30.45	" P.	87.197	.....	4.63	5.257	" P.
21	" 12	95.258	94.259	30.45	" P.	87.197	.....	8.38	5.257	" P.

Elevation of bottom of channel, back of bulkhead at experimental weir = 81.94 ft.
Height of crest of standard weir above bottom of channel of approach = 13.13 ft.
Length of crest of standard weir..... = 16.00 ft.
Length of crest of experimental weir, Nos. 1-19..... = 6.58 ft.
Length of crest of experimental weir, Nos. 20-21..... = 6.53 ft.

* Placed 8 ins. above the bottom.

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No. 885.

FILTRATION OF WATER FOR PUBLIC USE.

An informal Discussion at the Annual Convention, London, England, July 3d and 5th, 1900.

Subject for Discussion.*

"The several Processes now used for the Removal of Objectionable Matter; their Comparative Sanitary Effect, Cost and Reliability."

By Messrs. Rudolph Hering, George F. Deacon, George W. Fuller, Samuel Rideal, Henry Davey, J. N. Greene, Ad. Kemna, K. E. Hilgard, Walter Hunter, Philip H. Palmer, E. Perrett, Andrew Johnston, H. Alfred Roechling, Nicholas Simin and Rudolph Hering.

Rudolph Hering, M. Am. Soc. C. E.—When continually growing centers of population reach a point when their water supplies can no longer be obtained from springs or wells, it becomes necessary to utilize larger sources, such as rivers. Open streams, however, are often turbid and polluted, at least by the surface washing of rain water, and it becomes desirable to purify such water before use. Thus originated the practice of filtering public water supplies, and it is interesting to many of us to know that, within a few miles of where we are now gathered, the first large filters for this purpose were constructed by Mr. James Simpson, one of the former presidents of the Institution, in the year 1829, for the Chelsea Water Company. Mr. Hering.

* It is expected that this discussion will be continued in a subsequent volume of *Transactions*. It seemed desirable that, so far as possible, the discussion should be made available, notwithstanding the fact that Mr. Lindley's contribution, as well as those of several foreign engineers, who have signified their intention of discussing the subject, are not yet in hand.

Mr. Hering. In the early days the aim was to get a clear and palatable water. Since the discovery of the bacterial origin of many diseases, and the further discovery that such pathogenic bacteria may be introduced into our bodies through the use of water, a further and most important requisite is the elimination of disease germs also.

Filtration has, therefore, a triple duty to perform: It must render the water healthful, clear and pleasant to taste.

As might be expected, different lines of practice have been followed in different countries, and even in the same country, in the endeavor to obtain the most effective and most economical means of water purification. The result has been the development of several distinct processes.

At the last international gathering of engineers interested in this subject, during the World's Fair at Chicago, in 1893, the results of studies pursued at the experiment station of the Massachusetts State Board of Health, were made known to the world. These results contributed much to the knowledge of removing bacterial germs from water, and elevated the practice of filtration from an empirical to a scientific basis.

Since then, further substantial progress has been made on both sides of the Atlantic, but particularly through the extensive experiments made on a large scale at Louisville, Cincinnati and Pittsburg, which covered the conditions of water highly charged with suspended matter as well as polluted by sewage. These experiments included also a test of filters of another order than those previously investigated in Massachusetts.

Experience has not yet brought about a general agreement regarding the relative merits of these several filters and methods of purification now in use. It was hoped, therefore, that a discussion at this second international gathering of engineers might contribute greater clearness as to the controlling elements, and thus lead to a more rapid progress toward the desired end.

We may divide the methods of water purification now practiced substantially into two classes: One requires a large bed of fine sand through which the water is allowed to percolate, at the rate of, roughly, from 5 to 10 cu. ft. per day on 1 sq. ft. of filter surface, or, a column 2½ to 5 ins. in depth, per hour. The other, with a much smaller bed of sand or similar material, requires from, say, 200 to 400 cu. ft. per day to pass through 1 sq. ft. of filter surface, or, a column of water of from 8 to 16 ft. in depth per hour, which is, on the average, a rate 40 times faster than in the first case.

As the chief difference is this rate of speed, the two systems have been called, respectively, slow and rapid filtration. The appliances have also been called "sand" and "mechanical" filters, because sand is the chief element in the first case, while mechanical contrivances

form the chief element in the latter case. They have also been distinguished, respectively, as the English and American filters, from the countries where they were first developed. Mr. Hering.

For filtering public water supplies, there are still other appliances, of which the Fischer or Worms system and the Maignen system have been most prominent.

The Worms system requires the use of artificial tiles, through which the water is made to percolate. Experience, however, has shown that with a reasonably large yield of water the purity may be insufficient, or *vice versa*, the expense being rather great for any other than originally clear waters.

The Maignen system requires the use of asbestos pulp, which is caused to gradually form a layer or cover over the sand beds of slow filters and collect the sediment and bacteria. It is claimed that it greatly increases the safe rate of filtration per square foot. The cleansing is effected by simply rolling up this cover, which separates readily from the sand. With this contrivance no experience on a large scale has yet been had.

The slow or sand filters have been used almost exclusively in Europe. The principle upon which they act is a double one: First, there is a straining action, in keeping out of the passing water all coarse suspended matter and many of the bacteria, by the slimy sediment layer which forms upon the surface of the sand; and secondly, there is a further disappearance of the bacteria caused by an organic process going on while the water is percolating through the sand.

The efficiency of such a filter depends upon a number of conditions, the most important of which are:

1. The Size of the Sand Grains.—It is found that fine sand of uniform size, several feet in depth, gives the best results. It is further found that the number of bacteria in the effluent water increases with a decreasing thickness of sand bed and with an increasing size of sand grains.

2. The Rate of Filtration.—A rate of from 2 000 000 to 3 000 000 U. S. galls. per acre per day, or a column of water about 3 to 4½ ins. in depth per hour, is found most satisfactory, as a rule. The number of bacteria passing through the filter increases rapidly with the rate, which should, in any case, be kept uniform to get the best results, and to accomplish which requires the use of controlling devices.

3. Drainage.—The free withdrawal of the water from the underdrains, to secure the same rate in all parts of the filter.

4. Temperature.—The temperature of the water should not vary greatly, because it controls the pressure required to effect a given rate of filtration, and because the bacterial efficiency decreases with the temperature.

Mr. Hering. *5. The Age of the Filter Bed.*—New or freshly scraped filter beds give higher numbers of bacteria in the effluent water than old and well-compacted sand layers.

The slow system of filtration has been practiced according to two methods, the continuous and the intermittent. When the amount of dissolved oxygen contained in the water is sufficient for the necessary bacterial action, the continuous method is preferable, because it is simpler and cheaper. It is, therefore, the more common one. When the water is highly polluted, requiring a large amount of oxygen for the proper bacterial action, the intermittent system may be preferable, as in Lawrence, Mass., where the sand beds must be given a frequent aeration, and thereby receive a new supply of oxygen; or, when the water is highly polluted, a double continuous filtration may be practiced, as in several cities of Europe. Still another variation is practiced in Bremen, where the first filtrate coming from the filter just after it has been cleaned, and therefore generally not sufficiently pure, is passed through another filter, which has not been recently cleaned, before it enters the city's mains.

In climates where water freezes frequently during the winter, and to a depth of at least several inches, it is customary to cover the filters in order to obtain the best results. Where it is impracticable to do so, as was the case in Hamburg, special allowance must be made for the decreased bacterial activity in the colder water, and for the mechanical difficulties resulting from the ice.

When the water to be filtered has a high degree of turbidity, economy demands that it be allowed to deposit most of its suspended matter before it is run upon the filters. Settling basins are therefore a necessary adjunct to almost all filters for river water. When this contains finely divided clay, the particles of which are smaller than bacteria, it is necessary to add a coagulant to the water before it enters the settling basins. Unless there is a prior removal of this fine clay, the filter may allow the water to pass through without giving it a thorough clarification.

One day's settling is usually sufficient for the purpose, as by far the largest proportion of suspended matter, and all the coarser matter is removed within that period. While the custom to provide for about one day's settling prevails in America and Germany, it has been preferred in England to build storage reservoirs sufficient in size to hold water for a much longer period. The idea being that, for economical reasons, enough water should be stored for use during the time when a storm makes the river water very turbid.

A slow filter is cleaned by removing a thin layer of sand, $\frac{1}{2}$ to 1 in. in thickness. This upper layer contains not only the suspended matter deposited from the water, but also most of the bacteria.

The second class of filters, the so-called rapid or mechanical filters,

were first introduced in America, and are extensively used there. Mr. Hering.
They act on the principle of straining, by which the suspended matter and the bacteria are kept substantially on the surface of the filter material. The rapid rate of the passing water does not allow of much organic action below the surface, and requires also a rather coarse filtering material.

In order to hold back the fine matter held in suspension, such as clay particles and bacteria, coagulating substances are added to the water in definite quantities, depending chiefly upon the amount of lime and organic matter contained in the raw water. The resulting flocculent matter settles upon the surface of the filter bed, and then acts chiefly as a strainer, sufficiently fine to hold back the minute particles, including a percentage of bacteria, which, under favorable conditions, may be as large as that removed by the slow filters.

Through the investigations chiefly of George W. Fuller, Assoc. M. Am. Soc. C.E., the conditions for the successful operation of such filters are understood now much better than formerly. It is possible, now, to arrange their operation, so that, from a water of a given quality, fairly definite results can be obtained. The efficiency depends mainly upon the size and character of the grains, upon the rate of filtration, the constancy of this rate, the proper admixture of the necessary coagulants varying with the quality of the water, and upon the arrangements for properly withdrawing the filtered water.

Two kinds of rapid or mechanical filters have been used, those acting under the slight pressure of the water contained in an open filter, and those acting under a high pressure in a closed filter. Although it is in many cases convenient and economical, the high-pressure filter cannot give equally good bacterial results, because the delicate film of coagulant may be more readily ruptured. High-pressure filters, therefore, should be used only for waters which are not seriously polluted.

A rapid filter is cleaned by reversing the current and washing the entire mass of sand thoroughly with filtered water, for which numerous and more or less efficient mechanical devices have been used. This washing process requires from 3 to 5% of the filtered water, and an opportunity of disposing of the waste water in an unobjectionable manner.

In case the water is very turbid a preliminary settling is of advantage also with this class of filters.

From the foregoing necessarily very brief review of the chief characteristics of the two principal methods of filtration now in use, it will be gathered that they may both satisfactorily purify water under many ordinary conditions.

The criteria which should guide in the selection of one or the other method for a particular case are their relative sanitary effects

Mr. Hering. under expected conditions, the reliability of their operation, and their cost.

In these introductory remarks it is not the purpose to enter fully and in detail into a comparison of the merits and demerits of the several methods. It is hoped that those who join in the discussion may do so. The speaker will merely state his own conclusions derived from his practice.

The sanitary effect of a filtered water will be judged by the prevailing death and sick rates. We have abundant evidence that a well constructed and faithfully operated filter of the slow type can materially reduce typhoid fever and diarrhœal diseases. There is hardly a question that it will also remove the contagion of cholera. A similarly strong statement regarding the sanitary effect of rapid filters can, from lack of experience, hardly yet be made, though we know that the bacterial efficiency of both classes of filters can be made almost equally high, the slow filters being probably slightly more efficient.

As to the reliability of operation, it seems to the speaker that the slow filters should be given the preference. The great difference in the rate of filtration furnishes the reason for this conclusion. When there is a rupture in the coagulated material covering the filter—and such a rupture is likely to be produced by some irregularity—the amount of unpurified water passing through the filter is many times greater than in the case of a slow filter, where such a rupture, even when it does occur, cannot produce as harmful results

Regarding the other chances of mishaps, they are less serious and may result equally in both cases. The skill necessary to operate the two kinds of filters may also be considered as equal, if the best results are reached.

We may conclude, therefore, that, as regards efficiency and reliability, both methods can render satisfactory service, depending on the special conditions of the raw water, the degree of purity to be obtained and the care exercised in the operation.

If the raw water is highly polluted, slow filtration, or a double filtration, is to be preferred. In other cases, the decision will generally depend upon the cost.

Regarding the cost, it has been found that often the two methods are about equally expensive. The required investment is usually greater for slow filters, but the cost of operation is correspondingly less, and *vice versa*.

We may expect, therefore, that both the slow and rapid filters will be used in the future with good results in their respective spheres, and that the peculiar conditions of each locality will enable the engineer to decide as to which will give the greater satisfaction at the lower cost.

Mr. Deacon.

George F. Deacon,* M. Inst. C. E.—The speaker has listened to Mr. Hering's discussion and to the observations of Mr. Lindley,† with great pleasure, and feels that on this occasion some of the English hydraulic engineers ought to pay a tribute of admiration to the manner in which the Americans have recently taken up this subject. In England, sand filtration has been carried on to a larger extent than in any other country, and it might be said here to be practically the universal system for potable water. On the Continent, some special tests and experiments have been made, but the speaker thinks that they are all eclipsed by the very complete series of experiments conducted by the Massachusetts State Board of Health, by the Pittsburg Commission, and by certain other authorities in the United States, and which are now referred to as among the most important investigations which have been made on the subject. Only within the last few years has it been discovered what sand filtration really means, and there is still one element which is not completely settled or defined in the minds of all engineers concerned in the subject, viz., the part which the sand really plays in the process. This much is known: That the sand is not much more than a strainer, and that it serves as a support to a gelatinous film which forms on its surface, and which really constitutes the filtering medium. Opinions differ as to whether, when this filtering medium is doing its legitimate work, the sand beneath it is or is not giving anything more than a supporting assistance.

The speaker's view is that the sand does little or nothing in the way of actual filtration, but that it is nevertheless necessary in considerable thickness, to ensure complete filtration by the film resting upon it. A thin bed of sand will not ensure uniformly distributed perlocation, and unless the initial percolation through the sand is distributed with fair uniformity, it is clear that the surface film cannot in all parts do its duty equally well, and that where the resistance is least, the suspended matters in the water will be carried into the sand to a larger extent than elsewhere. That this is so may be seen in almost any filter bed which, when the water has been lowered, is carefully cut through with a spade. Parts of the bed are found to be discolored more than others, and it is noticeable that the discolored portions are not in vertical lines but in greatly inclined lines, often passing for several feet to one side and back again more than once before reaching the bottom of the sand. In many filter beds also where the sand has not been disturbed for several years, it has been observed that this feature is accompanied by an actual re-sorting of the grains of sand, the coarser grains being found in somewhat similar inclined zigzag lines. It might seem incredible to some engineers that such re-sorting could take place at so low an average velocity—not more than

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† Owing to Mr. Lindley's illness, his discussion could not be prepared for press. If possible, it will be issued in the next volume of *Transactions*.

Mr. Deacon. 4 or 5 ins. per hour—as that used in filtration; but the mystery disappears when it is considered that the weight of the sand grains when under water is reduced 40%, and that the mean velocity of 4 to 5 ins. per hour varies inversely as the space—the interstices of the sand—through which the water flows, and directly as the enormously lengthened path it traverses, while the maximum velocity at points in the mass of sand must greatly exceed the mean. Moreover, as a matter of fact, examination of old filter beds has shown conclusively that the devious paths taken by the lines of maximum velocity must still further very greatly increase that velocity, and must also leave places of almost absolute quiescence in which the finest grains will deposit.

At certain works, which he believed to be among the best managed in Great Britain, the speaker many years ago saw this feature so marked, that a vertical section of the sand gave the impression of the condition known in geology as "false bedding." He had lately seen the same works again, and, although in the interval the sand had been changed several times, exactly the same feature occurred. This was an extreme example, but the same thing might be seen in a modified degree in nearly every case. Practically, the water did not go down vertically or nearly vertically, and was just as erratic as a piece of paper let fall through the air.

Taking into consideration how extremely easily the resistance of the surface film might be broken, and the fact that in badly managed water-works it was sometimes purposely broken, it seems to the speaker that this tendency to irregular resistance of the sand, constantly increasing as the film grows older, is the explanation of the fact that a thin supporting bed does not in all cases give so good a result as a thick bed. No doubt very much depends upon the uniformity of the grains of sand, for it is perfectly clear that if all the grains are of the same size, re-sorting could not take place, and would make no difference if it did.

For these reasons, in his own practice he had increased the thickness of the bed of fine sand to 3 or 4 ft. to begin with. This left room for a reduction in the depth of 1 ft. 6 ins., or 2 ft. as a maximum, caused by the successive operations of scraping the surface. The period between the scrapings depends very much on the weather and the condition of the water, but on the average in England it is from 10 to 20 days.

There can be no doubt that, in one way or another, the surface film should be allowed to form before the water is filtered for use, and, if time can be afforded, it seems to be better to allow that film to form by sedimentation rather than by filtration, as immediate filtration after scraping always carries suspended matter into the sand.

The practice set out by Mr. Hering and by Mr. Lindley is very much the practice to which English engineers have lately attained. The

automatic regulation of the water to be filtered is most important, and in recent practice the speaker has always adopted it. He goes rather further than Mr. Lindley; and not only regulates the flow automatically, but measures it by a differentiating meter on a diagram, in such a manner that if any interference with the surface of the film takes place at any moment or in any place, it will be recorded immediately by the suddenly increased flow. Mr. Deacon.

Similarly, the rate of diminution of flow owing to increased resistance of the surface film is constantly recorded. The depth of water above the sand is regulated by a float, and remains practically constant, but the head below the filters is determined by passing the water over a stand-pipe in the clear-water tank having its overflow between 2 and 3 ft. below the level of the water in the filter beds. Within that limit the head is controlled by a valve between the filter and the stand-pipe. It is the attendant's business to set this valve daily, so that the meter will not record more than the prescribed maximum flow nor less than the prescribed minimum, and each such setting is duly recorded on the meter diagram at its appropriate time. These diagrams, which make known any irregularity, are transmitted to the water-works office periodically.

The use of some such automatic and restrictive records as these is probably the best possible safeguard against those sudden introductions of changes in the surface film which are known to occur, and which render possible the passage of bacterial life through the filter.

It is to be observed that a filtration plant thus controlled does not require a workman of superior intelligence to manage it. When once he understands that he is to keep the flow through each filter within the two limits prescribed on the diagram, there is no difficulty whatever; and if, from other causes, any irregularity should occur, it cannot escape the graphic record or fail to become known when the diagram reaches the office.

Of so-called mechanical filters, the class without coagulants is of little use, except for straining purposes. With coagulants used from day to day under intelligent direction, mechanical filters will, when in the best working condition, produce results nearly equal to those of sand filtration. But in practical working the speaker does not believe that this can be so. The proportioning of the coagulants cannot be automatically done in a perfect manner. The work has to be put into the hands of much more intelligent men than those required for sand filtration, and he knows of no means of securing anything like the same uniformly efficient conditions as those so readily obtained from properly constructed sand filters, provided with apparatus for recording automatically all the changes, whether prescribed or irregular.

Sand filters are nothing if not sure defenses against the passage of

Mr. Deacon. bacteria. In nine cases out of ten this is their principal object; and where it is so, the speaker is satisfied that for public supplies sand filtration, properly conducted, is better than any other known device.

Mr. Fuller. GEORGE W. FULLER, Assoc. M. Am. Soc. C. E.—For the engineer who has to consider water filtration problems in different parts of America, the speaker thinks there is no point more deeply impressed on his mind than that the waters from different sections of the country possess to a large degree a distinct individuality in reference to their purification. Having listened to the excellent *résumé* of this topic by Mr. Hering, the description of the Warsaw filters by Mr. Lindley,* and the discussion on English practice by Mr. Deacon, it is thought that it would be of interest to look at the subject for a few moments from the standpoint of the applicability of different methods of purification to different types of water.

One of the fundamental principles of water purification, as viewed from experience in America, is that the quality and the composition of the water to be filtered are factors of prime importance. This is particularly so with American surface waters, where there are wide ranges in character. In this discussion consideration is given only to surface waters, and not to ground-waters which may need treatment for the removal of iron. With the surface waters from different sections of the country, the most variable features, from the purification standpoint, are with reference to turbidity and color. By turbidity is meant the sand, silt, clay and those matters of an amorphous and organic nature which are contained in the water in suspension; and by color is meant the vegetable matter which is dissolved in the water, and which gives what is called by many a vegetable stain.

In the original application of water purification, improvement in the appearance of the water was, of course, the object sought. Within the last fifteen or twenty years the problem has changed, as is well known, and, fundamentally, the object now to be secured is the removal of those bacteria which are associated with the causation of disease. In looking at the problem from that point of view, it may be of interest to point out a difference which occurs between conditions in America and in Europe.

In Europe, particularly in England and in the western portion of the Continent, or, more accurately, with those filtered waters with which American engineers are more familiar, the speaker understands that it is generally true that if the water is filtered so as to eliminate the bacteria, the appearance of the water is satisfactory. That is to say, if the effluent is satisfactory from a hygienic standpoint, it is not necessary to give attention to the appearance of the filtered water. But in America the reverse of this would be true in numerous

* Owing to Mr. Lindley's illness, his discussion could not be prepared for press. If possible, it will be issued in the next volume of *Transactions*.

instances. In many places—not in the northeastern portion of the country, but in the South and Central West—the character of the waters is such that if the water after filtration is satisfactory in appearance, it is also substantially free from bacteria. This shows, in a general way, a difference, which perhaps is seldom appreciated, in the conditions in America and in Europe. The reason for this, of course, is that the clay particles found in the southern and western waters of America are much smaller in size than the bacteria. It is a fact that many of the clay particles in these waters are not more than 0.00001 in. in average diameter, or about one-tenth the size of the ordinary bacterium. Some of the clay particles are still smaller, and, without doubt, our estimates of their size are much limited by the present facilities for measurement under the microscope. Mr. Fuller.

While a filtered water, of course, should primarily be as free of bacteria as is practicable, it appears to the speaker that the appearance of the water should also be satisfactory, in order to fulfill requirements as found in many cases. In this connection, it is to be borne in mind that many water consumers judge the character of a water by its appearance; and, regardless of the hygienic side of the question, do not look with especial favor upon a water which is either turbid or colored to a considerable degree Public opinion in America appears to be undergoing a gradual but distinct change in this respect. Colored and turbid waters which seemed to be acceptable a few years ago are now complained of in many instances; and, in the opinion of the speaker, the development of the "spring water" trade and of house filters, is suggestive of conditions which water-works engineers must consider in the near future.

When turbid waters in America are to be purified for hygienic reasons, it seems to be the consensus of opinion that the purification works should yield an effluent which is also practically free from turbidity at all times. With waters containing much vegetable stain opinion varies widely. In fact, in America it would seem that the amount of permissible (dissolved) color in a first-class water is one of the most unsettled points at present, in connection with water supply and water purification. The permissible limit depends, of course, to a large degree, upon the point of view of the observer. Perhaps the most critical observers at present in America are the water consumers themselves, who judge the appearance of the water in bulk with a white background as they see it on their tables and in porcelain-lined bath tubs.

To decide, in a conservative and reasonable manner, upon the permissible amount of dissolved color in water for the future requirements in America is a task which is as difficult as it is desirable. So far as the speaker has been able to form a judgment upon this matter, he would be inclined to place the limit at 0.30 part of dissolved color

Mr. Fuller. by the platinum-cobalt standard. This amount of color is about half of that frequently seen in many waters in the eastern part of America, and about double the amount found in the southern and western waters after the removal of the suspended silt and clay.

Returning now to the character of the American waters, it may be said, in a general way, that there are three types, as viewed from the present standpoint of purification. There is, first, the water from those comparatively short rivers, and from lakes and ponds, which lie within the glacial drift formation, that is, the waters best known to us as those of New England and the northern Atlantic States. There is also another very distinct type of water which lies south of the glacial drift formation almost entirely, that is, the large rivers lying south of and including the Potomac, the Ohio, the Missouri, and extending from the Atlantic Ocean west to the Rocky Mountains. Between these types there is an intermediate class resembling in a measure the two; such waters as are found in the Delaware, the Susquehanna, the Allegheny, and rivers which lie along the dividing line between the drift formation and the section of country where the drift is absent.

To attempt to make specific comparisons of these types of water is a very difficult matter at the present time. In the first place, the amount of analytical evidence, with few exceptions, is very meager indeed, and especially with reference to results from samples which represent the normal and maximum conditions which the engineer must consider in the design of purification works. There is also another reason why these data are somewhat wanting in the significance they carry, and that is the lack of uniformity in the analytical methods used in obtaining the results. This latter point is becoming realized more and more clearly by engineers, and also by the analysts themselves. In passing, it may be of interest to state that an effort is being made to correct this state of affairs. At the last meeting of the American Public Health Association a section of Chemistry and Bacteriology was formed, and there was appointed under that section a Standing Committee to consider the question of Standard Uniform Methods of Water Analysis. Ultimately, it is hoped that some good may come of this.

Comparing with each other these general types of American surface waters in regard to the turbidity, or suspended matters which they contain, it may be said that those bodies of water lying wholly in the area of the glacial drift formation, such waters as that of the Merrimac, Connecticut, Hudson and Passaic Rivers, carry from about 5 to 15 parts per million of suspended matters as an annual average. This quantity is greatly exceeded at times of heavy floods, and in the case of the Merrimac River the amount of suspended matter has been known to exceed 1 000 parts per million. The conditions of the spring

freshet in this instance, however, were abnormal. With regard to the character of the turbidity in this type of water, it may be stated that, comparatively speaking, it is caused by fine silt, and not by clay. Accordingly, it is a type of water which is not very troublesome in its purification, as the silt can be removed quite readily by subsidence. Mr. Fuller.

The waters of the second class, those of the South and West, contain very large amounts of suspended matter. The annual average amounts in the Ohio River water have been found to be 230 and 350 parts per million at Cincinnati and Louisville, respectively. In the case of the Mississippi River water there are records showing the annual average amount of suspended matter at St. Louis to be about 1 000 parts per million, and about 560 parts at New Orleans. These records cover information obtained at different times during the past fifty years, and are by no means strictly comparable with each other. They are sufficient, however, to show that this general type of water contains roughly from twenty-five to fifty times as much suspended matter, on an average, as do the waters of the northern Atlantic seaboard in America, and also the English river waters as the latter are known to American engineers. From a practical purification standpoint the comparative difficulty in handling the very turbid river waters of the South and West is by no means adequately shown by the annual average amounts of suspended matter. There are several reasons for this. In the first place, the flood periods continue uninterruptedly for many weeks, and, in some instances, extend over several months, during which time the suspended matters vary very rapidly and continue normally in excess of the annual average, reaching at times more than 5 000 parts per million. Another feature of great importance in explaining the difficulty in purifying these very turbid waters is the fact that a considerable proportion, say about one-fourth, of the suspended matters consist of minute clay particles. The sub-microscopic nature of these particles has already been referred to by the speaker, and it may be further added that it is practically impossible to remove them by subsidence.

The third, or intermediate, class of water, such as found in the Delaware, Susquehanna and Allegheny, etc., contains suspended matters ranging apparently from about 25 to 75 parts per million, on an average. In duration of flood periods, proportion of clay particles and in the size of clay particles, this class of water stands intermediate between the two previously described. The evidence, however, shows that in some instances there are decided complications from coal dust and mine drainage.

With regard to the amount of dissolved color or vegetable stain contained in these general types of water, it may be said that many, but by no means all, waters of the first class contain more color than is considered by some to be satisfactory. To this class of waters of the

Mr. Fuller. glacial drift area there must be added swamp waters from all sections of the country. No attempt will be made to show in detail the general evidence as to the color of this type of water, partly because of lack of available comparable records, and partly because of the unsettled condition of the problem of what amount of color is permissible in a first-class public water supply.

The dissolved color in the waters of the second and third types is not a factor of practical significance, so far as the speaker has been able to learn. Undoubtedly, organic matter capable of producing a vegetable stain reaches these turbid waters, but it seems to be removed in the streams and settling basins. Apparently, it becomes united to the silt and clay particles (hydrosilicate of aluminum) and settles to the bottom with them. In a measure, this may be called filtration of silt and clay through water.

From this account of the conditions met with in America, it will be seen that the method of purification to be adopted in any instance must be adjusted to the requirements of the water under consideration. Further, the character of many surface waters in America is such that, in the design of purification works which shall successfully meet the requirements of the future, careful attention must be given, not only to the question of bacterial removal, but also to the decolorization and clarification of the water.

Of the methods of purification now considered in America, there are two important ones, as has been very clearly shown by Mr. Hering, who has described and compared their leading characteristics. The nomenclature used in this line of work in America is in quite a transitional and unsettled state. The term "filtration of water" was satisfactory so long as it was applied to waters which were not excessively colored or turbid. Since the subject has grown and included the consideration of the more complicated types of water, the term "purification of water" is considered by many to be more logical and comprehensive. For many waters, filtration by itself is inadequate, and there are required in connection therewith plain subsidence, coagulation, and supplementary subsidence. In some places the expression "method of filtration" has been replaced by "method" or "systems of purification." With regard to the two methods of filtration proper, they are variously spoken of as sand, or English, or slow filters, and as mechanical, or American, or rapid filters, respectively. Upon comparing the two methods of filtration when each is operating at a high degree of efficiency, it is found that there is a long series of differential features, of which the most important is the physical condition of the water as it enters the sand layer. To many it seems desirable to avoid a nomenclature which gives undue prominence to one superficial feature, and to use a name which implies a series of conditions requisite for first-class work. By these engineers the two

methods are called English and American filters, respectively, from the countries where they are first developed. Mr. Fuller.

Water purification on a practical scale in America is in its infancy. Ten years ago its application was almost nominal. During the past decade, however, considerable progress has been made in the small cities and towns. At present it may be said, in general terms, that there are about 20 English sand filters in service in America, having an area of about 17 acres, and yielding about 25 000 000 galls. of water as a daily average There are also in service about 160 American, or mechanical, filter plants, having a nominal capacity of about 250 000 000 galls. daily, but actually supplying a consumption of only about half this quantity.

With few exceptions, these purification plants by both methods are very small, and there is comparatively little information available, showing continuously in detail the efficiency and the cost of operation. Under these circumstances, American engineers are obliged to base their detailed knowledge of water purification, in a large measure, upon the results of European experience and of the careful investigations made on an experimental scale at Lawrence, Providence, Louisville, Pittsburg and Cincinnati. The European experience, in connection with the purification of waters comparable to those now filtered abroad, has been carefully followed in America for many years, beginning with the classical report made by the late James P. Kirkwood, M. Am. Soc. C. E., for the City of St. Louis. For the purification of the more difficult types of water, due to their excessive color or turbidity, dependence is now placed on the laws of purification as deduced by the special studies in the cities just named. Upon the more important points it is considered that this evidence is substantially correct for the types of water actually studied, and it is now being used as a guide in the design and construction of purification works for a number of the larger American cities.

Concerning the relative merits of the two leading methods of purification, the experience of the speaker has led him to the opinion that English sand filters, as a rule, are most satisfactory in point of efficiency and economy for those waters for which they are strictly applicable. Such waters may be defined as the non-clay-bearing waters, which are only slightly or moderately discolored by vegetable stain, and which are found in the first type of waters mentioned in this discussion. Although it is well known that these filters have decided limitations in the successful treatment of very colored and very turbid waters, it is a difficult matter at present to define satisfactorily the dividing line between those waters for which this method of purification is applicable or non-applicable. There are some cases in which this method is strictly applicable for the local water where it is probable that its adoption would be inadvisable on the ground of cost,

Mr. Fuller. owing to the area which it requires, thus necessitating an isolated location and an extra pumping of the water. Notwithstanding the fact that English sand filters are applicable for a far smaller proportion of the American water supplies than was generally considered to be the case a few years ago, it is the opinion of the speaker that they have a large field of usefulness in America, and that they will be adopted in many places within the next few years.

Referring to the construction and operation of English sand filters in America, it may be fairly said that the recently completed purification works at Albany, N. Y., designed by Allen Hazen, M. Am. Soc. C. E., and described in a recent paper* before this Society, may be taken as representative of the best current knowledge in this line, based both on European experience and American investigations. These filters differ much from the older filters of Europe, but do not differ very materially in the more vital points, when compared with the best modern filters abroad. Nevertheless, they have an individuality of their own. Among the more interesting and important features of the Albany filters, from the standpoint of efficiency and economy, are the arrangement of the floor with the thin layers of gravel to support the sand, and the means taken to prevent the passage of water between the walls and the sand.

The recent estimates made for English sand filters at a number of American cities are on the basis of an average capacity of 3 000 000 galls. (U. S.) per acre daily. In view of the fact that the maximum consumption in some places ranges from 25 to 50% above the average, during very hot and very cold weather, the rate would at times reach between 4 000 000 and 5 000 000 galls. per acre daily, even with moderately large distributing and compensating reservoirs. From present evidence this seems reasonably safe for very short periods, unless the raw water should be excessively polluted. In fact, it appeared from experimental evidence obtained several years ago at Lawrence and elsewhere, that, with some kinds of water, rates of 5 000 000 and even 7 500 000 galls. per acre could be safely used. More recently, however, the very careful studies made at Lawrence, with reference to the passage of *Bacillus Coli Communis*, the leading intestinal bacterium, indicate that these rates are too high for such waters as that of the Merrimac River. In the event that rates of filtration have to be increased materially, to meet unusual conditions of consumption, experience indicates that the changes should be made gradually and not suddenly. In fact, proper regulation of the rate of filtration is now universally recognized as an essential feature of successful filtration; and the introduction of devices to secure this end shows one of the most marked improvements in the new over the older filters.

There is some difference of opinion in America concerning the con-

* *Transactions*, Am. Soc. C. E., Vol. xliii, page 214.

ditions under which covers should be provided for English sand filters. A few years ago it was thought that this could be settled by the mean January temperature, on the basis that covers would be required where this temperature was at or below the freezing point. More detailed studies of temperature conditions, however, show that every few years there are exceptionally cold winters during which there are so many days in succession when it would not be practicable to scrape open filters, that the foregoing rule seems to be questionable. According to the opinion of many engineers, experience will ultimately show that covers are an advantage at nearly all points on and north of the Potomac and Ohio Rivers. If this should prove true, it would necessitate covers in nearly all the American territory where English sand filters now appear to be applicable. With regard to the effect which covers produce upon the bacterial efficiency of the filter, in connection with vegetable growths, etc., this is a point upon which there is very little or no evidence in America. Mr. Fuller.

With regard to the American or mechanical filters, it is well known that they are proprietory devices of comparatively recent origin. The more important patents, however, will expire shortly. Prior to 1885 these filters were used exclusively in the treatment of water for industrial purposes for the removal of vegetable stain and turbidity. Since that date they have been introduced for the purification of many public water supplies, and during the last few years progress has been very rapid. For the most part they have been installed for small works, not so much because they were necessarily the most applicable process for each water, but rather perhaps on account of the filter companies guaranteeing certain results with devices of fairly low first cost.

Up to 1895 very little was known of the hygienic efficiency or cost of operation of these filters. Naturally, they were not looked upon with favor by the great majority of engineers. During the last five years there has been a great increase in our knowledge of the leading features connected with these filters. This is particularly so in the detailed knowledge of coagulating processes, which are such a vital matter in this type of filter. Without coagulation one can expect from them no hygienic efficiency. As already stated, the proper degree of coagulation of the water as it enters the sand layer is the keynote to their successful work. When this is satisfactory, such factors as rate of filtration, size of sand grain, thickness of sand layer and detailed method of cleaning the sand layer, appear, within reasonable limits, to drop into comparative insignificance. At one time it was quite universally feared that the use of such coagulants as sulphate of alumina might result prejudicially to the health of those using water purified in this manner. As a result of most careful studies upon this point, it may now be stated that when the coagulant is applied in

Mr. Fuller. proper quantities in the presence of proper quantities of alkaline constituents of the water, there need be no fear of undecomposed chemical in the effluent. With this process it is imperative that the supervision should be such that at no time should the inadmissible result be met of applying more coagulant than can be completely decomposed. In this decomposition there is a conversion of the carbonates of lime and magnesia into the sulphates of these bases, with the liberation of a corresponding quantity of carbonic acid. The alumina itself is, of course, removed as a flocculent precipitate.

If the merits of filters of this type were to be judged from the general knowledge as to their efficiency, as now obtained in practice in the majority of cases, the indications are that it would not receive a rating of the highest grade. This seems to be due, in part, to a lack of the necessary skilled supervision at small plants, and, in part, to a desire to avoid the expense of applying a sufficient quantity of coagulant. From the results of extended investigations upon the subject, however, it is considered that sufficient knowledge is available to design and operate by this method plants which will give first-class results, when judged practically from the highest plane of excellence. From the added knowledge of tests of short duration upon recently constructed plants in practice, it is felt by the speaker that the laws of this process, so far as they have been formulated, are substantially sound and accurate.

Concerning the applicability of this method of purification, it is the judgment of the speaker that, for the non-clay-bearing waters, which are low in amount of dissolved color, it is, as a rule, less suitable than English sand filters. This is based on the present general evidence that for this type of water the English filters can give a thoroughly satisfactory effluent, and at a total cost equal to or slightly less than that of American filters; that the use of coagulants is to be avoided where not imperative; and that the use of coagulants for these waters is especially undesirable and exacting in the control of their application, due to their low alkalinity. There are, of course, exceptions to this general rule, such, for instance, as cases where the compact form of the American filters would permit of a location where extra pumping would be avoided, and where this could not be done with English filters.

The especial field of usefulness for the American filter is in connection with the purification of those waters which are either so highly colored, or so highly charged with sub-microscopic clay particles, that it is impossible to obtain satisfactory results without the use of coagulants. With waters which require a coagulant, either constantly or for the great majority of the time, the present evidence shows that, as a rule, the American system of purification is the better. For very turbid waters this system consists of plain subsidence, coagulation and sup-

plementary subsidence, and mechanical filters. For such waters the use of English sand filters would increase the total cost somewhat; they would give normally no higher grade of purification; they would be more difficult to operate; and, for the latter reason, there would be a tendency for them to yield a less satisfactory effluent at times when the river water would be most variable in character. Mr. Fuller.

In explanation of the point that, with English filters, purification plants for very turbid waters would be more difficult to operate, it may be said that the great rapidity with which the character of the water changes makes it very difficult, and in a measure impossible, to apply at all times just the proper quantity of coagulant at the inlet to the coagulating basin. Obviously, this would be kept as low as consistent with good efficiency, for the sake of economy. If it should be found that the application of coagulant was too small, it is possible with American filters to correct this by adding more coagulant as the water reaches the filters, thus maintaining the efficiency of the plant. With English filters this is not possible, because, if the coagulant were applied at the outlet of the coagulating basin, it would clog the surface of the sand very quickly, and put the entire plant out of service for some little time. From a practical standpoint, the speaker attaches great importance to this feature, which was repeatedly noted during the investigations at Cincinnati.

There has as yet been very little experience in the purification of waters containing large amounts of dissolved color. Judging from experience with the recently completed works at Norfolk, Va., it appears that the American method with mechanical filters is the better, for reasons similar to those stated for the clay-bearing waters.

Concerning the purification of those intermediate types of water, which for short periods are moderately colored or turbid, there are not sufficient data to draw any well-defined lines. A careful study of local conditions, with reference to the characteristics of the two methods of purification, can alone settle the matter properly for the present.

Samuel Rideal, D. Sc.*—The remarks on the American practice with mechanical filters have greatly interested the speaker. Speaking as an English chemist and bacteriologist, he thought that the lesson to be learned from this discussion was that English engineers should give these mechanical filters a trial in this country, parallel to the trials that American engineers were giving to the English method of sand filtration. Mr. Fuller's remarks seem to show that there is a future for mechanical filters in England, where recent practice has been to do away with the sand-filtered water supplies from rivers, and to go to the mountain and upland surface waters, many of which, containing vegetable coloring matter, would seem to be very amenable to this mechanical filtration. These waters frequently give rise to plumbism. With a Dr. Rideal.

* London.

Dr. Rideal. mechanical filter using alum (and lime, if the water is not sufficiently hard from carbonate of lime), the coagulant would correct that evil; so that with mechanical filtration, using alum with or without lime, we should effect the removal of the vegetable coloring matter and prevent plumbism at the same time. That our method of sand filtration seems generally to give a better bacterial purity than the mechanical filters, may be due to the fact that the latter have been worked at a much greater rate, and perhaps with the minimum quantity of coagulant, without due regard to the reduction of bacteria; a difference that doubtless could be removed.

Neither method of filtration can, however, be regarded as a satisfactory means for preventing the appearance of pathogenic germs in water. No matter how small the fraction of the organisms left, there might be among them some which might be injurious to health. Therefore, although either sand filtration or mechanical filtration is a very good line of defence, it is not the only line of defence upon which one should rely.

The speaker believes that the question of sterilizing water will soon become prominent, and that the removal of pathogenic organisms can only be effected satisfactorily and economically by heat sterilizing. He used the term "heat sterilizing" (a somewhat misleading phrase) purposely, because he wished to accentuate before a body of engineers that it is not a question of heat sterilization at all, but a temperature condition. The organisms are killed when they reach a certain temperature, and a maintenance of this condition does not necessarily involve the expenditure of any energy.

The problem of bringing a body of water to a high temperature, and keeping it at that temperature sufficiently long to ensure the death of the organisms present, is one which he believes would be worth while for engineers in this country and in the States to pay special attention to. The cost of bringing very small quantities of water—perhaps two or three pints per head of the population, instead of the 65 or 75 gallons—to a temperature which will ensure the killing off of the pathogenic organisms, cannot be very large. By a proper system of incoming and outcoming of the water, and possibly by a vacuum jacket such as is used for maintaining a very low temperature, the expenditure of the heat or energy, either gas or electric heating, which would be required for bringing about such local sterilization of water for potable purposes, would be very small indeed. Probably in the States as well as in this country legislation could be brought to bear in the same way that we now have by-laws for limiting the quantity of waste in flushing closets. In this way it would be possible for the municipalities to supply to every consumer a machine for producing sufficient sterilized water for potable purposes, while leaving the supplies and mains in their present condition.

HENRY DAVEY,* M. Inst. C. E.—The speaker thinks that a high tribute is due to Mr. Hering for his discussion, in which the case is stated very clearly, and in which opinions are expressed which seem perfectly in accordance with the facts of practice. On this question, and referring only to the English practice, or what has been termed the sand filter, the speaker, who for many years has had charge of very large filter beds dealing with large quantities of very impure river water which it is necessary to filter and bring into a proper state for domestic use, has very little to say, because the general principles have been so thoroughly stated that he could only repeat what has been already said, or go into minute questions of detail which would not be profitable. Mr. Davey.

He would, however, like to emphasize what has already been said. The question of sedimentation is a highly important one. If storage reservoirs can be made sufficiently capacious, the quality of the water may be so improved that the work on the filters is reduced. In the sand filter, as has been pointed out already, it is only the thin layer of the surface that does the work, and the object is to maintain that thin stratum in a quiescent state in order that the bacteriological effects may properly take place. The rate of filtration, then, is an important point, because time must be given for these bacteriological operations. Four inches per hour has been named, and that, according to the speaker's experience, is good practice in England; but, as Mr. Deacon has intimated, the longer the filter works, the less will be the rate. If an average of 4 ins. is taken, it may be too rapid to begin with. We know that when the water is put on the filter immediately after the filter has been cleansed, that there is very little effect—the water goes through. The filter should be worked for the first two or three days at an exceedingly low rate in order to produce the film that is necessary without letting many bacteria go through. When the filters are newly made they should be worked very slowly indeed to get the best results. Much depends on the class of water, but, from the speaker's experience, 4 ins. per hour should be the maximum with waters that are high in bacteria.

Although a thin layer of sand is all that is necessary to maintain the surface film, yet a considerable thickness is desirable, for the reasons which have been mentioned by Mr. Deacon. The speaker has found that in a section of the filter bed having a depth of sand of say 4 ft., there are streaks of pollution through the sand, showing that if that had been very much thinner much polluted matter would have gone through. He thinks it is a safe thing to have a great depth of sand, but, as has been already pointed out, it should be sand and not pebbles and gravel, which are only useful as a support. An important point which has given the speaker considerable trouble is that, unless

* Engineer, Birmingham, England, Water-Works.

Mr. Darey. the inlet to the filter is properly constructed, the current of water around it disturbs the sand, and the water at once finds its way down through without the surface film being formed. It is not necessary to go into the actual construction, but merely to mention that the distribution of the water over the filter and on the filter, in such a way that the surface is not disturbed, is an important point to be taken into consideration.

Mr. Greene. J. N. GREENE, M. Am. Soc. C. E.—The speaker would like to ask some of the learned gentlemen who have discussed these questions, and who have had experience in filtration, to explain how to eliminate from the filtered water, after it has gone through the filter, all the filthy things, such as the drainings from cesspools, stables, dye houses, tanneries and everything that borders the streams which flow into the river; how can all those impurities, which are held in solution, be eliminated, and a good water be secured by filtering?

The real secret of this evil of consumption of foul water lies often, and, indeed, in most cases, in the short-sighted, make-shift way in which communities treat this, one of the most important subjects connected with their life and health. In numerous instances, all over this and other countries, large cities and small communities, upon this "good-enough-for-the-present" plan of doing things, have expended large sums upon impure water improvement, knowing it to be so at the start, and growing worse every year; and, even with such experiences, go on and enlarge the same, at great outlays of money, which they do only to increase an existing evil, instead of putting the knife to the root and expending their money for new plant to secure pure, or approximately pure, water. Philadelphia, Pa., and Bangor, Me., are striking examples of this class. In short, the speaker does not believe it is practicable or possible to obtain pure water by filtering water which is loaded with impurities held in solution when the filtering process is begun.

The surest way to get pure water is to go further and spend more money and get water that is pure and fit to use, without the artificial and never-satisfactory process of filtering.

Dr. Kemna. DR. AD. KEMNA.*—It is apparently easy to point out many shortcomings in the introductory discussion, and practically this is what has been attempted by the various speakers. But on closer examination it appears that all that has been said may be found in a very concise form in Mr. Hering's discussion, the interesting discussions being but instances of the general principles laid down as a basis.

There is, however, one important point which it is necessary to emphasize. The action of sand filtration is essentially a biological one. The mechanical action of straining is obvious; but, as microbes are much smaller than the interstices between the grains, this action alone does

*Of Antwerp, Belgium.

not account for their retention, and it is absolutely inadequate for the reduction of the dissolved organic matter. Various theories have been proposed; the one which considers reduction as a fermentation is consistent with the greatest number and the greatest variety of facts. Dr. Kemna

So, for example, an apparent contradiction can be easily explained. Animal pollution is easily reduced by filtration, while we have it on the authority of Mr. Fuller that vegetable color is extremely hard to remove. Now we know from our school chemistry that vegetable organic compounds are much more stable than animal nitrogenous contamination; the filter masters the one and fails with the other. Besides, microbes do not act on all substances; in racemic acid the tartrate will be destroyed and the right remains unaltered; similar cases are likely to occur with the mixture we call organic matter in water.

Negative proofs can be given of that biological action. In moderate climates, the bacterial efficiency of sand filtration is over 99%; but in colder climates, like Warsaw and Odessa, only 96% is reached, and, according to Simin (Moscow), the reduction is as low as 60 per cent.

There was a spell of sharp frost over Western Europe in December, 1899. The cold had not set in three days before the filters of Antwerp gave insufficient bacterial and chemical purification. The size of a pin's head of the slimy upper layer gives in the field of the microscope generally over 200 living organisms, chiefly diatoms; during the cold there were 4 or 5, and it seems reasonable to admit that their vital activity also is greatly diminished. A chemical process (iron and marble) was applied with good results to insure the quality of the supply.

Another proof is the interesting fact that at Louisville, according to Mr. Fuller, ordinary sand filtration does not work; the high turbidity of the Ohio water shuts out the light, which is essential to the development of diatoms.

The sand filter has been, and is still, in many places, a very simple thing; it is a heap of sand; water is poured on the top and extracted from underneath; it is worked by a foreman, who knows, of course, how much water he pumps on the filters, but he cannot always tell how much each particular filter is doing, its speed, etc.; generally the filters are left to settle that between themselves. This is the acme of simplicity.

But now compare with what has been said up to the present time. At the intake end, Mr. Lindley* wants a very elaborate arrangement for subsidence, to be worked differently according to meteorological data. In summer the water is made to travel downward; in winter it is to take the reverse course. The admission to the filters is regulated by automatic processes so as to keep a constant level above the sand. The filter itself, as described in 1889† presented many special features,

* Owing to Mr. Lindley's illness, his discussion could not be prepared for press. If possible, it will be issued in the next volume of *Transactions*.

† Congrès pour l'Utilisation des Eaux Fluviales, à Paris.

Dr. Kemna. namely, a series of layers of gravel systematically arranged. The water is not allowed to leave the filter without having to pass a special chamber, with automatic machinery to regulate its flow. But its troubles are not over yet, for now comes in Dr. Rideal, who says, in effect, that filtration does very well; it takes out 99.9 and a series of 9's of microbes, but that he wants to kill by heat the last one. All these various processes added together make up a long list of complicated appliances.

It has been said that facts are the only things considered by engineers. But facts are nothing by themselves and they get a bearing only in conjunction with a theory to explain them; if it is contended that there is no theory, that means only that there is no avowed theory and that we are led by an unconscious one. Can there be any doubt that an outspoken theory, which can be discussed, is to be preferred? The very common belief that theory is of no practical avail and is at the best an intellectual luxury is utterly wrong; it is an inquiry as to causes of phenomena and the reasons for things, and is therefore an absolute necessity. No better example can be adduced than sand filtration itself, which made no progress during fifty years, and was rapidly modified and perfected as soon as the biological theory gave the right explanation. It will pay to examine by the light of our theoretical knowledge, as far as possible, the various complications which have been proposed in the ordinary sand filter. But at the same time we must keep in mind that there are practical limits.

Taking up again the course of the water, we meet at first sedimentation, relied upon in England for improving the quality. Direct oxygenation is the explanation which probably is in the minds of most engineers; but experimental proof of this action could not be given. Mr. Fuller has suggested another cause—the subtraction of dissolved organic matter by the suspended clay. Then there is vegetable life favored by light, which also does a good deal. But it is still an open question whether the amount of preliminary purification is worth the money invested in large reservoirs.

When the water is kept in constant motion, Mr. Lindley's scheme seems to be the best to ensure the utilization of the full volume of the reservoir for decantation. The walls and partitions necessary to make the different channels are a small extra cost when the reservoir is covered, as it must be under a climate like that of Warsaw. But when ordinary open reservoirs are used, the surplus expenditure becomes comparatively large. It can be avoided altogether, however, by subdividing the reservoir into separate sections, successively drawn upon, and in which there will be no flow at all; many observations seem to show that there is a better settling with a few hours' complete rest, than with a long run even at a very slow speed. This very simple theoretical consideration enables us to consider the complications of the intake, not as an absolute advantage in all cases.

A great deal of simplification has been done by Mr. Lindley himself as to the gravel. The only part the gravel has to play is to give a support to the sand and an exit for the water; and as soon as this is done efficiently, nothing more is wanted. Dr. Kemna.

The thickness of the sand is a much-discussed question; the small size of the grain is all-important for the mechanical action of straining; the thickness of the bed is all-important with the physical theory of surface action, very popular some decades ago, when chemistry told that dissolved matter was abstracted from the water. But it is an undisputed fact that quartz has only a very limited power of adhesion, quite unlike that which is so marked with aluminum compounds. In the light of the biological theory, the sand is something like the gravel—a support to the superficial living layer. The well-ascertained fact that thick filters give an effluent of more constant bacteriological composition has received a simple mechanical explanation from Allen Hazen, M. Am. Soc. C. E.; if the superficial layer is broken in some places, a thick bed of sand allows of only a small direct flow, and contamination is kept down to a minimum.

As the reduction of dissolved organic matter is a fermentation, some time must be allowed for it; the retention of the microbes is a sort of compromise between the erosive mechanical action of running water and the stickiness of the stuff coating the sand grains; for these two reasons the rate of filtration must be kept within certain limits. The 4 ins. per hour, or about 2.5 m. per day, is the limit fixed by the German Imperial Health Office; it is a limit, and, as the engineer of Birmingham has very well pointed out, that speed cannot always be given. The pace of a man walking is, say, 1 m., but that cannot be expected from a child. When a filter has just been scraped it is in its infancy and must be worked very gently at first. As soon as the filtering layer has formed, the speed may be increased, and, according to experience at Antwerp, may be much more than the 4 ins. per hour and still give good results.

The suggestion of Dr. Rideal to sterilize the water is based on the fact that some epidemic diseases are caused by water-borne microbes. It is obvious that absolute security can only be obtained by complete sterilization, but the practice of fifty years' filtration proves that the risk of contamination is an exceedingly small one. A heating apparatus for thousands of cubic meters means a large expense; a special canalization for a few liters of water a day is another big item. It is a question whether that very small increase of security is worth the extra expense, the more so as the temperature of 70° Cent., mentioned by Dr. Rideal, will certainly not kill all the spores, some of which are very resistant.

The discussion has wavered between English and American filters. In both cases sand is applied, but that is the only point they have in

Dr. Kemna. common. In both cases this point is unimportant, the sand being only a support—in the ordinary filter a support to a layer of algæ and clay in which biological actions are prominent—in the mechanical filter, merely for straining out the coagulum. Ordinary sand filtration is not at all an artificial process. We cannot order life to multiply, and the microbes and algæ to work in a special way; we are completely at their mercy. In the mechanical filter a chemical compound of known composition and predetermined reactions is used which can be adequately dosed. The comparison between the two processes is really an antithetic one.

It is useless to emphasize the necessity of purifying all superficial waters when used for distribution. It is only a question of cost, depending, of course, on the amount of water to be treated; so that the possibility of bettering a supply is intimately connected with the stoppage of leaks and wasteful use. This appears to be very large in England and absolutely enormous in America. It causes large expenditure to provide water, the tenth part of which only is really used. It also increases the amount of sewage, making here again purification more costly, and contaminating the water-course, sometimes the very river from which the supply is derived. Generally, people are shrewd enough not to pump the same water over and over again, but send it down the river to the town below. In the first case, the course of the water is a circulus of life; in the second case, it is a spiral.

Referring to the remarks of Mr. Walter Hunter, the speaker feels bound to say that the abandonment of bacteriological analyses for the water-supply of six million of people is a step in the wrong direction.

Answering Mr. Greene, it must be admitted that pollution cannot be kept out, and must therefore be dealt with. This the filter does, and the fact should be emphasized, that sand filtration is not an artificial or a theoretical process, that it is a natural process, especially because that process is exactly like what goes on in the purification of springs, which invalidates the sentimental argument against purified river water. A clear, beautiful, wholesome spring is something which comes out of the ground; but before coming out it was in; and before being in it had to come in from above as rain water through the air. The first stage of entrance, viz., the percolation through the upper layers of soil is a stage of enormous pollution, much greater than that of any stream. But lower down all is taken out by the natural agencies of the soil and by a series of very complicated chemical, physical and bacterial processes, exactly the same as in the surface layer of a sand filter. Engineers and scientists who have charge of a sand filter cannot claim that they do the work; it is the filter which does it for them, and does it very well.

K. E. Hilgard, M. Am. Soc. C. E.—Most of the methods of filtration which have been described by previous speakers apply to river water. Just before the speaker came to London he had occasion to visit a small plant in Zurich, Switzerland, where sand filtration is applied to non-turbid lake water and has been carried on for a great many years. Within the last few years the rate of filtration has decreased very remarkably, owing to considerable increase of impurities in the water. This was due to the fast growth and consequent spreading of the city, the rapid settling and building up and providing of drainage along both shores of the entire lake, as well as to the increase of navigation. To these factors may in no small measure be due the rapid increase, down to an increased depth below the surface, of animal and vegetable "plankton" (living organisms) especially of crustaceæ in the clear water of that beautifully situated lake. Mr. Hilgard.

The water is drawn through a submerged intake at some distance from the shore. The filters, which years ago could be used for periods of three or four weeks without needing to be cleaned, had to be cleaned at least once a week within the last few years, so that, for this reason alone, quite a number of the several filter chambers are simultaneously and at regular intervals out of useful operation, because it requires one entire day to clean one chamber and as during two days after the cleaning, until a new and effective "film" has formed on the filter bed, the water from this chamber is not deemed fit for use, it is run through the waste.

In order to increase the rate of filtration and thereby the capacity of the entire plant, first as an experiment and now in actual usage, a primary or preliminary filter, in German called a "Vor-filter" was installed, the result being that the original filters proper, or now the final or "pure-filters," have not to be cleaned again within a period of from four to five weeks, thus permitting a greater number of them to be simultaneously in useful operation.

The description of the cleaning of these preliminary filters may be of some interest, as it is effected, not only by reversing the current of water, but by mixing with the latter a current of air which is forced up through the filter beds of the "Vor-filters," a current of water alone not having given any satisfaction. This preliminary filter retains all the coarse matter which, in this case, though abounding in quantity, is of small size, as the Lake of Zurich forms a large and deep natural settling basin. The lake is a little over 20 miles long, with an average width of about $1\frac{1}{4}$ miles, and into it empty a number of mountain streams which thus indirectly provide the City of Zurich with water.

In spite of the rather immense size of this settling basin, the silt and other mineral impurities, as well as the suspended organic matter, are not sufficiently settled. By means of the preliminary filters so much of this matter is retained that the final filters have to do much

Mr. Hilgard. less work than before; thus the rate of filtration is increased very materially and much better water is obtained. The rate of filtration is now 7 m. in 24 hours, which corresponds very closely to 1 ft. per hour. The filtering plant is not very large, and it provides a supply which varies between 6 000 000 and 7 000 000 galls. a day. There are ten filtering chambers altogether.

One matter of interest in connection with this plant is the fact that these "Vor-filters," not considered necessary when the plant was first

General Plan of Zurich Filter Plant (10 chambers)

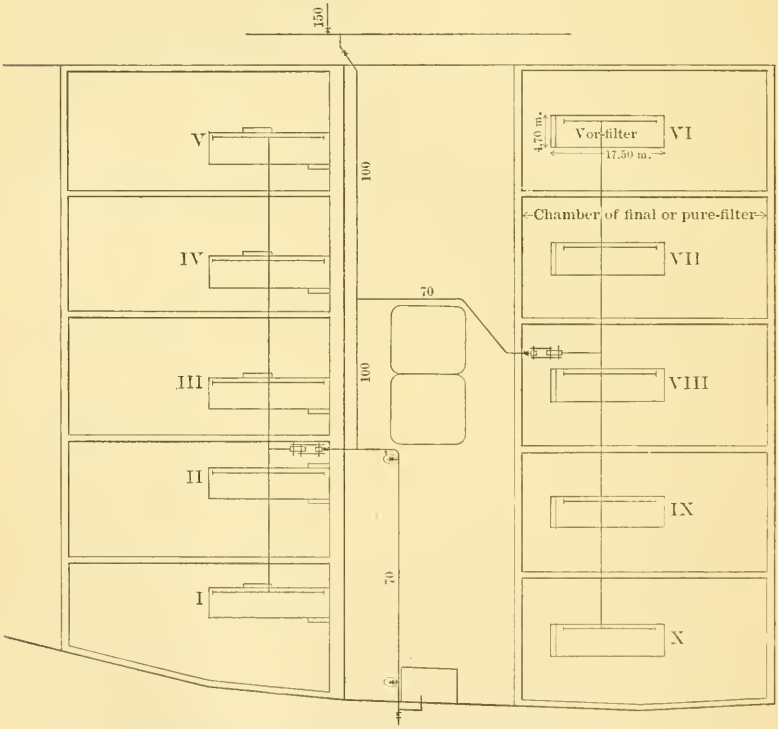

Fig. 1

built, and being, therefore, an afterthought, are designed and built on novel lines of construction, the so-called "Hennebique" system of *Béton armé* (reinforced concrete). The "Vor-filter" basins are supported above the final or "pure-filters," between the pillars of the vaulted chambers, by means of very thin side-walls, forming, with the intervening floors, sets of box girders constructed by this special method. The construction of these filters is shown in Figs. 1 to 7.

Reference is made in particular to Fig. 4 (plan of a primary or

Primary Filters for the Zurich Water-Tanks.

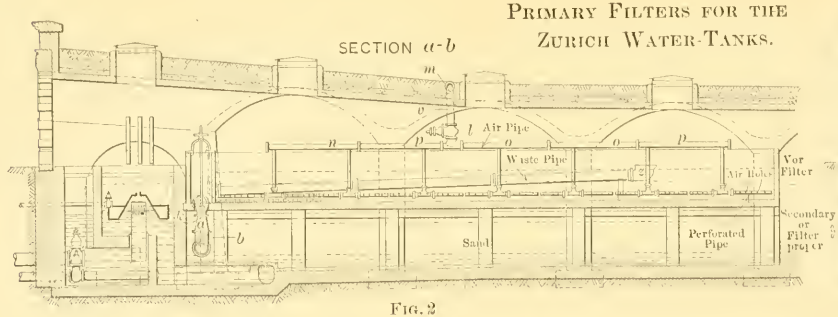

Fig. 2

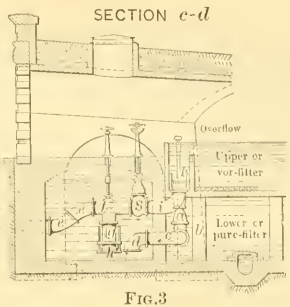

Fig. 3

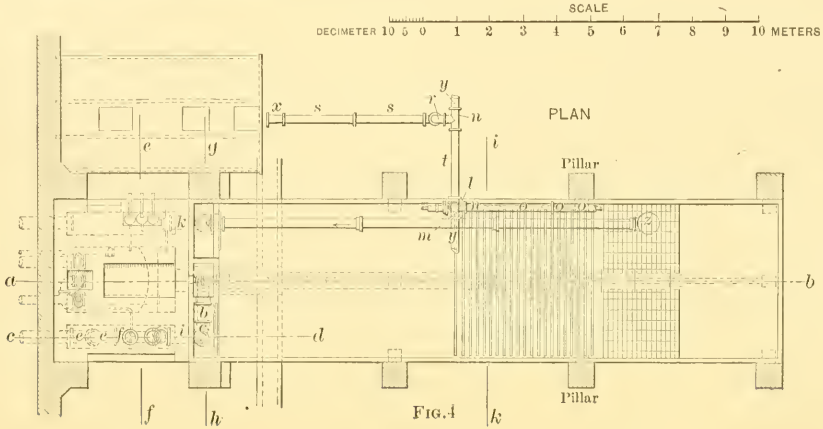

Fig. 4

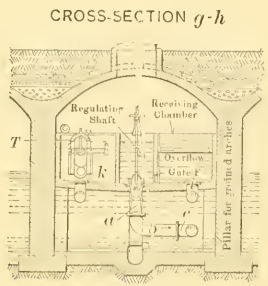

Fig. 5

Mr. Hilgard. Vor-filter basin) and Figs. 2, 3, 5, 6, and 7 (sections through primary and secondary or main filters).

The raw lake water enters through parts *c*, *e* and *i* (Fig. 3) from below into the upper "receiving" chamber and by overflow across the dividing partition into the upper or "Vor-filter" chamber. After percolation through the primary filter bed and draining through the hollow tile floor it collects in the main under-drain in the center and passes into the "regulating" chamber from which it descends through the "regulating" valve and parts *a*, *b*, *c* and *d* into the lower receiving chamber and from the latter into the lower filter basin, containing only the original main or pure-filter beds. In case the raw lake water is found to be sufficiently pure and need not pass through the Vor-filters—not improbable at certain periods of the year—it can enter the pure-filter directly by closing the valve *S* and opening the valve in the trumpet *g*.

In order to clean the "Vor-filter" the gate *F*, in the partition between the upper "receiving" and the "regulating" chambers, is

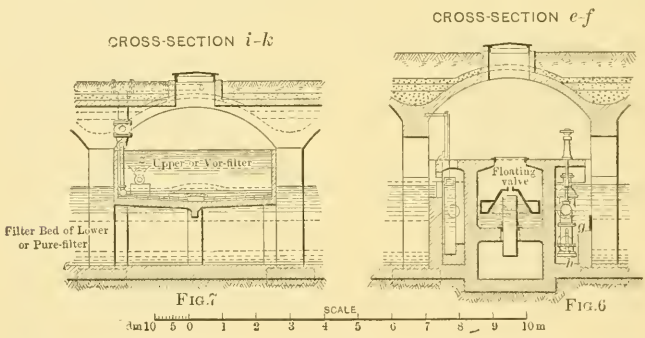

FIG. 7. FIG. 6.

raised so as to allow the raw water to enter from the former through the latter directly into the collecting drain and to rise from below the hollow tile floor up through the filter beds. At the same time two coupled ventilators, driven by a small turbine are set in motion, and force air through the perforated pipe (Fig. 2), with a pressure equal to a column of water of 1 m. (about 1½ lbs. per square inch), up through the sand, while the top layer of the filter bed is being thoroughly stirred and scraped by men with suitable tools. The muddy water is then drawn off through the trumpet-shaped intake *z*, and the connecting waste pipe, and enters a collecting chamber, from which, by raising the gate *T* (Fig. 5), it is wasted into the river near by. This operation is repeated two or three times, and the Vor-filters are thus cleaned every day in the forenoon. In the course of a year the bed in the Vor-filters has to be completely renewed.

Figs. 2 and 6 show the floating tube through which the filtere water in the pure-water chamber discharges into the main, and by

gravity flows to the pumping station. The float regulates the discharge according to the varying water level due to the varying loss of head in the filter. At the pumping station the filtered water is forced into the different "zonen-reservoirs." Mr. Hilgard.

The cost of the entire plant (consisting of ten vaulted chambers, containing the main filters and Vor-filters) complete (exclusive of grounds), was about 120 francs per square meter of sand surface, equal to about $20 per square yard. The total area of the Vor-filters is one-tenth of that of the pure filters. The total area of both is 7 000 sq. m.

The annual cost of operation, exclusive of interest, depreciation of tools and sinking fund charges, that is, labor only, exceeds but little the sum of 20 000 francs, making the cost of filtering (labor only) about $1.60 to $1.80 per million gallons, based on the present total annual consumption of 11 000 000 to 12 000 000 cu. m., which corresponds to an average daily consumption of filtered water of about 200 liters = 45 galls. per head for 160 000 inhabitants, or a total daily average of nearly 7 500 000 galls. The average scale of wages for labor in the United States and Switzerland being in the proportion of about $2\frac{1}{4}$ to 1, and the average buying power of these wages being in the proportion of between 2 and $4\frac{1}{2}$ francs to one dollar for cost of dwelling, clothing and food, respectively.

Most of the public fountains in the city are supplied from springs, a considerable number of which are owned, and others are being constantly bought up, by the city, even beyond the cantonal boundaries; while most of the water used for industrial purposes (water-power for small industries and trades) is not filtered, being supplied from a special high-pressure reservoir into which raw lake water is pumped at night.

Within the last year the "bacteriological results"* showed an average number of bacteria in 1 cu. cm. of water as follows:

	Quarterly average.	Annual average.	Absolute Number Found.	
			Max.	Min.
In raw lake water	700 to 2 060	1 260	3 700	320
In effluent from pure-filter	16 to 36	27	150	5
In filtered water taken from service pipes in buildings	100 to 190	140	2 360	8

	I quarter.	II quarter.	III quarter.	IV quarter of year.	Temperatures.	
					Max.	Min.
Average temperature of water (Celsius.)	4°	9°	14½°	11½°	15°	3½°
Average temperature of water (Fahr.)	39°	48°	58°	53°	59°	38°

* Annual report of H. Peter, Engineer of Water supply, for 1899.

Mr. Hunter. Walter Hunter,* M. Inst. C. E.—The speaker agrees with Mr. Davey and other speakers as to the desirability of sedimentation before filtration, as no doubt it eases the working of the filters, but wishes at present to deal especially with Dr. Rideal's remark, to the effect that it is necessary to take the last odd microbe out of the water. He recollects very well being told by Mr. Hawksley, whose name is so well known in the water world, that when he was at the meeting of the British Association at Bristol two years ago, there was a learned scientist there of the same profession as the learned doctor himself, who stated that he had put his children on sterilized food and water and they immediately began to droop. He found it was absolutely necessary to return to ordinary food and ordinary water in order that they might thrive. That is a practical illustration. The speaker thinks we owe an immense deal to the chemist and bacteriologist, who set us up an ideal to which we could strive to attain; but still he believes they sometimes go beyond what it is possible to attain in practice, and also beyond that which is really desirable. He has noticed that sometimes a sample of water has been termed inferior in quality, because it happened to have just double the quantity of certain chemical matters contained in another sample. Take organic carbon, for instance. One sample had 0.162 in 100 000 parts, and another sample 0.309, the difference being 0.147 in 100 000 parts, or $\frac{1}{680}$ to $\frac{1}{272}$. Again, organic nitrogen in one sample was 0.028 in 100 000 parts, and in another 0.033, a difference of 0.005 in 100 000 parts, or $\frac{1}{20\,000}$ to $\frac{1}{20\,000}$. These are figures one cannot easily grasp, and are really infinitesimal, both samples being equally good for all practical purposes.

After all, the great testimony as to the quality of any water is found in the statistics in regard to the health of the community supplied. The admirable way in which the water supply of London is conducted, for instance, can be so shown. When the death rate in the Metropolis is, as it was a week or two ago, something between 12 and 13 per thousand, and when the average for the year continues from year to year to be something between 17 and 19, and when, too, the very small amount of enteric fever which we have in this enormous city is noticed, the inference that the filtration of the water is carried out in a very admirable way, and that the community has no need to fear in regard to the quality of that water, seems fully justified. The Thames has been wonderfully improved by the action of the Thames Conservancy during the past few years. The raw water taken from the river is at least 33% better than it was. We are able to deal with it absolutely well by filtration, taking out something like from 98 to 99%, or 99.999% of the microbes, and the chemical analyses, as made by Sir William Crookes and Professor Dewar, and also by the government analyst, Doctor Thorpe, are most satisfactory. The speaker desires to add that

* Engineering Director, Grand Junction Water-Works Company, of London.

Doctor Thorpe, the new chemical examiner for London, has dropped the system of microbial examination which was adopted by the late Sir Edward Frankland, and confines his attention to chemical analysis. Mr. Hunter.

PHILIP H. PALMER,* M. Inst. C. E. (by letter).—Ten years ago the writer was induced to try Polarite for the filtration of drinking water. Up to that time he had used the ordinary filter bed, made with a 12-in. layer of fine sand, a 12-in. layer of coarse sand, a 9-in. layer of fine shingle, and coarser material, making a total thickness of 4 ft. The rate of filtration varied, according to the state of the bed, from 470 galls. per square yard per 24 hours up as to much as 650 galls. Mr. Palmer.

The drinking water supplied to Hastings is derived from deep wells, and is highly charged with oxide of iron, both in suspension and solution. This is brought out, on contact with the air, and forms a fairly thick deposit of oxide of iron on the surface of the filter bed. The writer tried an experiment with Polarite with very good results; a small filter bed was made with 6 ins. of coarse-grained sand and 12 ins. of Polarite underneath it. The rate of filtration was increased enormously, and the quality of the water after filtration was as good as when filtered at a slower rate. He then constructed at some of the filter beds a large receiving chamber having a screen between it and the distributing channels to the filter beds filled with Polarite. These screens were about 4 ft. high, and the space between them was 2 ft. wide. The water, after being discharged into the receiving basin, passed through the Polarite before being turned upon the ordinary filter bed. After passing through this material the water was freed of a very large part of the iron in suspension, and analyses showed that a very great improvement had been made in the quality of the water.

All the water supplied to Hastings, amounting to considerably over 1 000 000 galls. a day, is now filtered through Polarite; and at present the writer is erecting a battery of six mechanical filters, each capable of effectually filtering 10 000 galls. per hour. These filters are 9 ft. in height and 6 ft. 6 ins. in diameter. They are made of steel plates, properly riveted and stayed, and can be worked at a pressure of from 5 to 40 or 50 lbs. per square inch. The filtering material is Polarite, and the results are far beyond any which have yet come to the writer's knowledge. A recent analysis of water passed through a similar filter is shown on the following page.

Filters of a similar kind are at work at Southwold, in Essex, and at Leighton Buzzard. The results obtained at the latter place are almost extraordinary. The water there is charged with iron to such an extent as to be very perceptibly inky in taste, but after filtration through Polarite in a mechanical filter, at the rate of 8 000 galls. per hour, this taste is entirely removed. The water, which is very cloudy before filtration, is rendered perfectly clear, and about 90% of the iron is removed.

* Water-works Engineer to the Corporation of Hastings.

Mr. Palmer.

	Unfiltered.	Filtered.
Appearance	Turbid.	Clear.
Color	Greenish blue.	Greenish blue.
Smell	None.	None.
Deposit	Appreciable quantity.	None.
Nitrous acid	None.	None.
Poisonous metals	None.	None.
Hardness before boiling	$10\frac{1}{2}$° Clark.	$11\frac{1}{2}$° Clark.
" after "	$4\frac{3}{4}$° "	$5\frac{2}{3}$° "
In grains per gallon.		
Oxygen absorbed from permanganate in 4 minutes	0.008	0.007
Oxygen absorbed from permanganate in 4 hours	0.030	0.029
Total dry residue	22.96	26.04
Color of residue	Brownish.	White.
Behavior of residue on ignition	Blackens markedly, burns off readily.	Darkens very slightly, burns off readily.
Nitric acid	Minute trace.	Trace.
Ammonia	0.0019	0.0000
Albuminoid ammonia	0.0022	0.0022
Iron in solution	0.035	Very minute trace.
Deposit, chiefly oxide of iron	0.98	.....

After an experience of ten years in the use of this material, and having experimented and tried other methods of filtration, the writer is of the opinion that it forms one of the best filtering mediums it is possible to obtain. Water can be filtered through it perfectly, and at a rate which is almost impossible with most materials. It can easily be cleaned, and does not appreciably lose its bulk through cleansing.

The following are analyses of water after having been filtered through Polarite:

Appearance	Clear.	Clear.	Clear.	Clear.
Color	Very pale blue.	Pale greenish blue.	Pale bluish	Blue.
Smell	Inodorous.	None.	None.	None
Deposit	None.	None.	None.	None.
Poisonous metals	None.	.....	.....	.....
Phosphoric acid	Minute trace.	.....	.....	.....
Nitrous acid	None.	.....	.....	.....
Hardness before boiling.	$9\frac{1}{2}$° Clark.	$10\frac{1}{2}$° Clark.	10° Clark.	10° Clark.
Hardness after boiling..	5° Clark.	.....	.....	.....
In Grains per gallon.				
Oxygen absorbed from permanganate	.....	0.010	0.017	0.009
Total dry residue	27.44	.....	.....	.....
Color of residue	White.	.....	.....	.....
Behavior of residue on ignition	Does not blacken perceptibly.	.....	.....	.....
Chlorine	7.14.	7.14	7.28	7.98
Nitric acid	Trace.	Trace.	Trace.	Minute trace.
Ammonia	0.0000	0.0000	0.0000	0.0000
Albuminoid ammonia...	0.0000	0.0023	0.0014	0.0007
Micro-organisms per cubic centimeter.				
Total	5	12	6	12
Liquefying	None.	0	2	0
Coli	None.	0	0	0

E. Perrett, Assoc. M. Inst. C. E. (by letter).—The writer was interested, a few years ago, in the design of some filtering apparatus for a town of 30 000 inhabitants. Six filters were at first supplied, and shortly afterward twelve more of a similar kind were provided, so that their satisfactory performance may be assumed. Mr. Perrett.

The filters were tanks, 5 ft. square, made of cast-iron plates, and were in sets of three. A few inches above the bottom was placed a perforated false bottom, supporting a wire-work frame which carried the filtering material, coarse-grained charcoal about 12 ins. deep. Between the false bottom and the bottom plate of the tank was placed a branched perforated pipe. Above the filtering material was placed a wire gauze drum provided with an outlet pipe for the dirty water and refuse.

When filtering, the water was brought in above the material, and, passing down through it, was filtered and escaped through a suitable outlet.

When cleaning became necessary, or rather, advisable, the outlet was shut off, the dirty-water outlet was opened, and compressed air at a pressure of about 6 lbs. was passed through the perforated pipes and up through the dirty filtering material, thoroughly disturbing it and washing away the dirt through the drum. The water was at times very dirty, but was not impregnated with sewage. The rate of filtration was about 120 galls. per foot per hour. The time occupied in cleaning was about 7 or 8 minutes for each filter every 10 or 12 hours.

When filters capable of dealing readily with muddy water are used, settling tanks can advantageously be dispened with.

The usual rate of filtering through ordinary sand filters is about 3 galls. per foot per hour, the filters being cleaned every 3 weeks. By cleaning every 12 hours a very high rate of filtration is possible, the maximum being maintained.

Andrew Johnston, M. Inst. C. E.—The discussion upon this subject appears to have been selected at a most appropriate time, when it is remembered how large a percentage of our losses in South Africa is due to diseases very greatly attributable to contaminated water. Mr. Johnston.

For some time the speaker has been conducting experiments with the water of the River Ouse, in Yorkshire, and for that purpose has had constructed two filters, each having an area of 4½ sq. yds. One filter is composed of sand and gravel only, and the other has a layer of polarite, 1 ft. 9 ins. thick, between sand and gravel, each filter having a total depth of 5 ft.

The water treated contains in storm time much fine clayey matter, and has a strong yellow or peaty color, due mainly to organic contamination.

Mr. Johnston. The sand filter reduced the color slightly, while the polarite removed it completely. Both filters, of course, removed all suspended matter as well as reduced the bacteria to one per cubic centimeter.

The following are the analytical results, in parts per 100 000 of the important constituents.

	Water from subsiding reservoir.	Water from sand filter.	Water from polarite filter.
Free ammonia	0.01672	0.0084	Trace only.
Albuminoid ammonia	0.0194	0.0096	0.0078
Oxygen absorbed in 4 hours	0.214	0.095	0.032
Bacteria	312	1	1

The sand filter was working at the rate of 480 galls. per square yard per 24 hours, and the polarite filter at the rate of 2 444 galls. per square yard per 24 hours, or, in other words, the polarite filter was doing 500% more work than the sand filter, and giving a higher degree of purification.

Comparing the foregoing results with those given by Mr. Lindley, in his admirable description of his elaborate system at Warsaw, it is found that the rate of filtration, in the speaker's experiments, as compared with his, is as follows:

The speaker's sand filter was working at the rate of 2 607 liters per meter per 24 hours, and the polarite filter was working at the rate of 13 273 liters per meter per 24 hours, as against the Warsaw maximum rate of 87 liters per meter. It should be stated, however, that the speaker first causes the water for the polarite filters to pass through a mechanical polarite clarifier, working at the rate of 27 000 galls. per square yard per 24 hours.

No coagulant is used, as it is against English practice to use any chemicals in water for drinking purposes.

Although it may be argued, and to some extent truly, that to compare the results of experiments extending over a few months with those obtained by Mr. Lindley in experiments lasting for a number of years is not fair, yet the speaker does not think such an argument will affect this subject, because he is able to state, from the experience of other engineers, that equally good results are obtained at waterworks where polarite has been used for many years. At the West Hants Water-Works, Christchurch (England), the supply is taken direct from the River Avon, is first passed through polarite automatic clarifiers, which remove the suspended matter, and is then, by simple arrangements, sprayed upon polarite filters. One feature of the method at Christchurch is that the use of the clarifier enables subsi-

dence reservoirs to be dispensed with, thus effecting a great saving in cost. Mr. Johnston.

The following is an extract from the Government Official Report of Dr. F. St. George Mivart, to the English Local Government Board, on the "Sanitary Circumstances and Administration of the Borough of Christchurch, South Hants," and shows the pollution to which the River Avon is subject above the intake of the water-works:

"The River Avon, from which the supply is now wholly derived, rises in Wiltshire, in the neighborhood of Devizes. I do not propose here to deal with pollutions which enter it in the earlier part of its course. On its banks, in the vicinity of Christchurch, are a number of considerable aggregations of population, most of which, as I understand, get rid of their sewage by discharging it more or less directly to the river a few miles above the intake. My inspection of the river did not extend beyond that portion of it in the immediate neighborhood of the Borough, and hereabouts I noticed that pollution is possible, by reason of pig-keeping upon a large scale, carried on close to the river, about a quarter of a mile above the intake, and from the keeping of animals and poultry by the tenant of the mill at the site of the water-works."

Notwithstanding the fact that the River Avon cannot be looked upon as an ideal supply, yet, so successful is the filtration process adopted that the water as it issues from the polarite filter is of excellent quality, and is shown by the following analyses to be brought into a condition equal to that of the purest well waters of this country.

	Kent Deep Well. Analysis by Dr. P. Frankland, December, 1898.	West Hants, Filtered. Analysis by Dr. A. Angell. December, 1898.
	Grains per gallon.	
Free ammonia	Not any.	Not any.
Albuminoid ammonia	0.005	0.006
Nitrogen as nitrates and nitrites	0.311	0.191
Chlorine	1.75	1.05
Total solids dried at 212° Fahr.	28.98	20.44
Total hardness	18.6°	11.0°
Permanent hardness	4.8	3.1
Removable by boiling	13.8	7.9

The polarite filters at Christchurch were laid down in 1895, and continue to work as satisfactorily now as when first put into operation.

At Hastings* (England), polarite filters have been used for ten years on the public water supply, and the following are analyses and report by the Official Chemist on two samples of Hastings water purified by polarite, and taken from two different filtering stations:

* Population supplied with water, about 80 000.

Mr. Johnston.

" Appearance	Clear.	Clear.
Color	Colorless.	Colorless.
Hardness before boiling	11,3° Clark.	11,7° Clark.
	Grains per Gallon.	
	No. 1 Sample.	No. 2 Sample.
Oxygen absorbed from permanganate	0,008	0,008
Chlorine	5,32	7,14
Nitric acid	0,21	0,12
Ammonia	0,0019	0,0000
Albuminoid ammonia	0,0000	0,0000."

"These waters are, chemically speaking, of great organic purity."

At Reading * (England), the whole of the public water supply is purified by polarite, and the following chemical analyses and bacteriological results, extracted from a report to the Water-Works Committee, show the effect of this method of filtration.

"The results of the analysis are expressed in parts per 100 000, and are as follows:

Chlorine	1,30
Nitrous acid	0,00
Nitric acid (N_2O_5)	0,46
Phosphoric acid	Very faint trace.
Sulphuric acid (SO_3)	0,03
Total solids	23,92
Free ammonia	0,0000
Albuminoid ammonia	0,0092
Oxygen absorbed from permanganate at 80° in ¼ hour,	0,0441
" " " " " 4 hours {	0,0895
Temporary hardness	15,2
Permanent hardness	4,6
Total hardness	19,8
Lead and copper	Absent.
Odor at 100° Fahr.	None.
Color and appearance in 2-ft. tube	Pale, green, clear.
Deposit	Slight. Microscopical examination satisfactory.

"Number of colonies of micro-organisms in the river water, 1 200.

"Number of colonies of micro-organisms in the filtered water, 8.

"The micro-organisms in both the river and the filtered water were considerably less numerous than they were last month, and over 99% of those existing in the former had been removed by filtration.

"The results of the chemical and bacteriological analyses are satisfactory, and show that the filters were working well."

Mr. Roechling. H. Alfred Roechling, M. Inst. C. E. (by letter).—The writer is very sorry not to have had the pleasure of joining in the oral discussion, but urgent unforeseen business had prevented him from doing so. He is glad, however, to make a few written observations, as he is of opinion that the exchange of views between brother engineers in different countries is very desirable and tends in the end to break down that barrier of apparent indifference which at times seems to exist.

He will confine his remarks to a point in sand filtration, which is frequently overlooked.

* Population supplied with water, about 80 000.

Mr. Roechling.

When Simpson first introduced sand filtration at the Chelsea Works on a large scale, about the year 1829, he had no idea in what manner the purification of water was brought about in this process; he imitated Nature, and, as Nature's own processes are generally simplicity and perfection, he succeeded. Since then this process has become popular in all countries, and its application has extended from year to year, but its merits were only instinctively guessed and were not exposed and explained scientifically until within the last 10 or 15 years, when, chiefly through the means supplied by Koch, it had become possible to ascertain the organized matter in water, *i. e.*, the germs, microbes, micro-organisms, or whatever name we may give these minute beings, which belong to the vegetable kingdom.

The results then obtained clearly proved that the process of purification in sand filtration was chiefly brought about by physical or mechanical agencies, *i. e.*, the suspended matters, both organized and anorganized (living and dead) were removed, but the chemical composition of the water was but little affected.

How did this take place?

The slimy skin, which formed sooner or later on the surface of every sand filter, when surface-derived waters were being filtered, acted as a very fine strainer in catching and retaining the suspended matters in the water. By degrees this skin became so exceedingly retentive that it would not even allow the liquid to pass through, and then the filter had to be cleaned.

But if there was an almost impermeable layer gradually forming on the sand surface, how was the air to pass through, without which the micro-organic life could not carry on its work in the pores of the sand, gradually splitting up organic matters and changing them into mineral substances? And even if the process of mineralizing organic matters were not entirely due to biological agencies, it was pretty well established that in the absence of oxygen these changes soon came to a standstill.

When it is remembered that sand filters are now being filled from below, and that every care is taken to drive out the air contained in the mass of the sand; when it is further kept in mind that afterward an impermeable skin is gradually formed on the surface, it is not surprising to find the results previously stated, viz., the removal from the water of all suspended matters, but no great improvement in the chemical composition of the water.

The writer had been asked how he explained a slight chemical improvement which was noticeable with most waters, and he had stated that these small chemical changes went on until the oxygen contained in the sand and water was used up.

Totally different to the sand filtration of drinking water was the filtration of sewage or any other impure liquid through land, which pro-

Mr. Roechling. cess the writer had named the natural self-purification of sewage. Here it was of the utmost importance to prevent a crust forming on the surface of the land, so as to admit plenty of air to the micro-organic life in the soil, which carried on the benevolent work of mineralizing organic waste matters.

The writer has not time to go into the very interesting difference between water and sewage filters, and will only point out that the conditions of work in them are reversed, hence the results are reversed also. In the filtrate from sand filters there are but few germs and no chemical improvement; in the effluent from sewage filters there are millions of germs and a decided chemical improvement.

In conclusion, the writer is of opinion that the raw material for a public supply for drinking purposes should be chemically and otherwise as free from objection as possible, so as to reduce the difficulties and the costs of purification, because it is a lengthy and expensive process to use first a filter constructed on the same principles as a sewage filter and afterward a water filter, as we now understand them.

Mr. Simin. NICHOLAS SIMIN, Esq.* (by letter).—Moscow will soon have to decide upon the method of filtering daily 45 000 000 U. S. galls. of river water, the quantity necessary for its municipal water supply. This circumstance was the cause of the writer's journey from Moscow to the United States of America in the autumn of 1897, when the so-called "American method of water purification" or "the rapid sand filtration with coagulation" had just begun to receive a scientific sanction in America.

The purification of water by subsidence and with the aid of coagulation by alum has been practiced since ancient times, and its good results in making the most turbid waters clear as crystal are well known. The modern "American method of water purification" is but an improvement of the older one by changing the process from a slow to a rapid one.

During the first ten years of its development in the United States it was not under scientific control, and was advocated merely on commercial grounds. The great advantage, however, of a method of rapid filtration with coagulation caused several series of well-arranged scientific investigations to be made, and their results led to material improvements.

The first of these investigations was made in the year 1893 by Edmund B. Weston, M. Am. Soc. C. E., in the City of Providence, R. I. In 1896 and 1897 more extensive investigations were undertaken in Louisville, through the initiative of Charles Hermany, M. Am. Soc. C. E., and directed by G. W. Fuller, Assoc. M. Am. Soc. C. E., who gave to the science of water purification a great literary work without equal. At the same time independent scientific tests of mechanical

* Chief Engineer of the Moscow Water-Works.

filters were made in Lorain, Ohio, by the State Board of Health, under the direction of Allen Hazen, M. Am. Soc. C. E. Mr. Simin.

At the close of his work in Louisville in 1898, Mr. Fuller made further studies in Cincinnati, Ohio, where the importance of subsidence, with and without coagulation before filtration, was especially well demonstrated. In 1897 the City of Pittsburg, Pa., made parallel experiments with slow and rapid filters, under the general direction of Mr. Hazen. In 1899 Mr. Weston again experimented in East Providence, R. I., and in Norfolk, Va., and in March, 1900, Col. Miller terminated a comparative test of slow and rapid filters in Washington, D. C., resulting in a recommendation to adopt the rapid method of water purification for 60 000 000 galls. daily.

All these scientific investigations combine to prove that "rapid or mechanical filters" are able to give good results; and although Mr. Hazen recommended slow filters for Pittsburg, Pa., this did not prevent him from stating that the rapid filters were also able to purify water, and to remove its coloring matter to a greater extent than possible with slow sand filters.

At the time, when the writer was still under the inspiration of his inspection of mechanical filter plants in the United States, when he was familiar only with Mr. Weston's work in Providence, and could have had but a superficial idea of Mr. Fuller's work in Louisville, he expressed his opinion* that no obstacle would be able to stop the spread of the "American rapid method of water purification." Now, since all the above-mentioned scientific investigations are in public possession, the writer cannot do otherwise than reaffirm that opinion.

The writer now wishes to glance at the matter from a Russian standpoint.

The method of water purification with slow filters has been used in Europe about seventy years, and it has proven that good results can be obtained by it. At the same time they are not always obtained. In Russia not a single case of slow sand filters can be considered as being wholly successful. The best filters of this kind were built in Warsaw by the talented Mr. Lindley, and they left the water turbid during the session of the second Convention of the Russian Water-Works Association, held in 1895. The slow sand filters in Odessa, also very well constructed by English engineers, work at the rate of only 3 ins. per hour, and yet cases of unsatisfactory effluent occur often. In St. Petersburg the slow sand filters work with too great a velocity, and their average bacterial efficiency does not amount to 60 per cent. In several other Russian cities slow sand filters are also made to work at too great velocities, on account of their insufficient size, and, therefore, give bad results. The writer believes that such cases of lack of

* *The Engineering News*, March 24th; July 21st, 1898.
The Engineering Record, March 19th; July 23d, 1898.
Fire and Water, Vol. xxiii, No. 12.

Mr. Simin. success occur also in Western Europe, where English slow filters are used at many city water-works.

American mechanical filters are almost unknown in municipal water-works in Europe, and many superstitions exist regarding them. Systematic scientific investigations of the "American method of filtration with coagulation and sedimentation" have not as yet been made here. The investigations now undertaken by the Moscow municipality, after the example of Louisville and Pittsburg, should, therefore, be considered as the first made in Europe.

The water is taken from the Moscow River, within the city, where it is considerably polluted by sewage and factory refuse, and has a very changeable character.* Many special investigations, already made at the Moscow experimental filter plant, have cleared the subject, and the results in general agree with those obtained in America. Last March and April, when the stage of the river was high, and when the water was very turbid, the mechanical "Jewell" gravity filter removed 97 to 99.8% of the bacteria, and the filtered water during the whole time was quite clear. Sulphate of alumina, in quantities from 4 to 6 grains per gallon, was used as the coagulant, such large quantities being necessary to obtain satisfactory bacterial results.

Although these investigations are intended to extend over one whole year, they cannot be considered as final, because they do not indicate the results which would be obtained if the water is taken at the proposed new intake for the city water supply, to be located 30 miles above the city. The water there contains but about 400 bacteria per cubic centimeter, and therefore will require much less coagulant for its purification. On that account it is proposed to continue the investigations at the place where the water for the future supply will actually be taken, and to make parallel experiments with mechanical and slow sand filters.

If the experiments should be limited to those now being made in Moscow, the writer is not sure that they would secure the adoption of mechanical filters for the Moscow water supply. The citizens are not yet accustomed to the idea that it is possible to dispense with slow sand filters for water purification. If the writer continues the experiments for another year, he will have more data, and believes it will then be easier to remove the prejudice against the American method of water purification.

The writer feels certain in his conclusion that the most reliable, the most comfortable, the easiest and cheapest method, and the one best adapted to control scientifically the purification of the Moscow city water supply, is the American method of rapid filtration with coagulation of organic matter.

*The Moscow River water, at the point where it is taken to the filters, contains from 25 000 to 2 500 000 bacteria per cubic centimeter; on the average, about 200 000.

Mr. Simin.

The reasons are as follows: In the daily practice of water purification one should certainly give preference to a plant, the sanitary operation of which can be more easily subjected to scientific control. Such a control is more readily obtained by the American rapid filters than by the English filters, for the following reasons:

1. If the normal operation of an American plant is somewhat disturbed, it is very easy to restore it by washing and coagulation. After a ten-minute washing it is possible in three minutes to receive a good effluent, while the effluent of the English slow filters cannot be quite satisfactory until 3 to 5 days after cleaning, during which time the filtered water must be allowed to run to waste. In case the effluent suddenly appears turbid, the filtered water must also run to waste until it clears itself. In this sense it is not possible to control a slow sand filter.

2. In countries having severe winters, the American method of water purification costs much less than the method requiring covered slow sand filters. (The cost of filtration by means of rapid filters and of open slow filters is about the same.)

3. By the American method it is possible, not only to make the water clear and to remove the bacteria from it, but also to remove the color caused by vegetable pigments and other organic matter. Slow filtration removes such coloring matter to a much smaller extent, which is an important and insurmountable defect of that system.

4. In the case of American filters an increase in the rate of filtration does not cause such a noticeable depreciation in the quality of the effluent, as in the case of English filters. At the Moscow experimental plant a satisfactory effluent is obtained with a rate of 150 ins. per hour, and it remains practically the same up to a rate of 250 ins. per hour.

5. Occupying over fifty times as much horizontal area as American filters of the same capacity, the English filters meet with serious difficulties when there are no suitable pieces of ground upon which to locate them.

6. The method of cleaning the English filters is complicated and inconvenient. The sand has to be removed by hand and periodically restored by hand. The washing of the sand by machines presents difficulties during freezing weather. The washing of mechanical filters is easy and quick, and obviates taking the sand out of the filters. The rapidity of washing obviates also the large reserve filtering areas of the slow method.

7. The existing English slow filters make it too easy to increase the quantity of filtered water, and thus to avoid the erection of new filters. To do this it is only necessary to increase the velocity or rate of filtration and to dispense with the wasting of the first effluent. Such a practice, however, is detrimental to the quality of the effluent water. With the use of mechanical filters an increase of the quantity of the

Mr. Simin. filtered water is much more easily and cheaply obtained, by adding a small supplementary rapid filter than a large English filter. Nor is it necessary to waste such great quantities of the first effluent. Furthermore, the increase of velocity of filtration does not affect the quality of the effluent to the same extent as in the case of English filters.

8. In many cases American filters offer a possibility of uniting the process of filtration with that of softening the water or of freeing it from iron. With the English filters this double effect is not attainable.

The greatest hindrance to the adoption of the American method is the fact that it requires a coagulation, that is to say, the introduction of chemicals into the water. It is often said that the public is afraid of the sanitary effects of a chemical treatment of the water which it is obliged to drink. The writer sees no justification in such fears. Mankind often uses chemical processes for various purposes, and to a great extent, so that the micro-chemical process of coagulation, when the water is subsequently to be filtered, must not be regarded as anything out of the way. The writer thinks that the spread of the American method at city water-works, when subjected to a good sanitary control, may be considered even as a sign of an increasing public culture and public reliance upon a triumphant science.

During the last few years America has made many scientific and practical investigations of rapid water purification. Not a single one of them has undermined its reputation, on the contrary, all of them have strengthened it.

America has investigated also the English or slow method of filtration, and has proved that it also may give good results.

The science of water purification is much indebted to America and to its literature for the studies made of both the English and American methods. Europe did not do as much for the English method alone as America did for both.

The writer thinks, however, that the American investigations on slow filtration cannot wholly serve as a basis, as regards its daily practical working, because during the investigations the filters are always placed in a more favorable condition than that which they are likely to get in practice. Only when made on large filter plants in daily use, and when free from idealization, are such experiments able to show clearly all the merits and defects in their true light. Our present water-works literature hardly yet contains a record of any such investigations as regards English filters. The writer feels sure that such investigations, if systematic and thorough, would clear the way for the further spread of the American method. Therefore, although insisting on the use of this method for purifying the new Moscow water supply, the writer recommended that one filter with a daily capacity

of 500 000 vedro (1 625 000 U. S. galls.) should be arranged according to the English method with all its improvements, and that such a filter should work under the best practicable conditions simultaneously with the mechanical filters. Mr. Simin.

If the writer succeeds in having an American filter plant of 45 000 000 U. S. galls. daily capacity, adopted for the Moscow water supply, he would take care that the English filter plant, which it is proposed to build for 1 625 000 galls. per 24 hours, should remain for permanent use, so that investigations of both systems could be made under permanent scientific control and on an equal footing.

The investigations made in the United States of America have thrown much light upon the method of rapid water purification. Let us hope that a similar light may be thrown upon the English method of slow filtration, which, almost exclusively for 70 years, has served in European water-works practice.

Rudolph Hering, M. Am. Soc. C. E. (by letter).—In reviewing the discussion on the subject of the filtration of water for public use, for the first time presented for international discussion, it is gratifying to observe so little diversity of opinion on so important a matter, and that the positions taken in the opening remarks have been quite generally sustained. Mr. Hering.

Mr. Fuller's reminder that, when speaking of the entire process, the expression "water purification" should be used instead of "water filtration," is rational and proper, because the word filtration does not always comprise the whole process. The chief point made in his discussion refers to the necessity of applying different methods to different types of water, because these greatly vary, particularly in America, as regards suspended matter causing turbidity, and as regards dissolved vegetal matter causing color. This fact, the more closely it is analyzed, will, the writer believes, gradually brush aside the remaining differences of opinion regarding the relative merits of the slow and rapid processes.

It is unfortunate that the opinions are not unanimous regarding the proper names by which these two processes are distinguished. Besides "slow and rapid," we say "English and American," and we also say "sand and mechanical" filters.

The latter pair of names seems to be the least descriptive, because sand is the chief material in both systems, and both have now mechanical contrivances to regulate the flow and also to clean the sand. Further, the writer does not think that the nationality of the first of its kind is sufficiently descriptive for a true and permanent distinction. We are not sure but that some essential improvements will be made in some other country than that in which the process originated, and the name will then have but little meaning. The writer has, therefore, used the terms "slow and rapid," because the quantity of water passing through a

Mr. Hering. unit of area in the two classes shows decidedly the most distinctive of all the characteristic features of the two processes, and one which is inherent to the process, and will of necessity be permanent. The cost per million gallons filtered, for the average case, is about the same, in both processes. The means adopted for retaining the bacteria are essentially different, and are more expensive in rapid than in slow filters, while, in balancing this greater expense, the former permit the water to pass with a correspondingly greater speed. As yet, we seem to have no single word to characterize these different means for retaining the bacteria, the only other essential distinguishing feature between the two systems.

Mr. Fuller makes an important distinction between the waters of Western Europe, which resemble the waters of New England, and those chiefly of the Mississippi Basin. In the former the principal object is to remove bacteria, because, after such removal, the water is generally clear—the other chief requirement. In the latter case the principal object is to remove the fine particles of suspended clay, and thus to clarify the water, because then the bacteria, which are much larger, will of necessity also be removed.

The vegetal stain is independent of these considerations, and, with our present knowledge, seems to be a matter of æsthetics rather than sanitation. It, therefore, represents a question of finance rather than hygiene, and the removal of this color seems to be governed by the question as to whether or not the community is willing to pay for it, much as the individual decides whether he will drink champagne or claret.

The character of the finest suspended matter may be so different, as Mr. Fuller points out, as to radically affect the method of treatment. If it is silt, it consists largely of silicates and vegetal matter, most of which is readily removed by subsidence. If it is clay, subsidence removes only the grosser particles, while the finer ones can be removed only by coagulants.

The use of coagulants is still much objected to by the medical profession, but it seems to be largely due to a lack of knowledge of the details of the process. The coagulating material need not pass through the filter. Its main object is to retain the bacteria and suspended matter, and to remove color. In doing this it changes its chemical constitution. Only when the quantity added is unnecessarily great, or when the alkalinity of the water is insufficient, both of which contingencies may be avoided or corrected, does it escape into the filtered water, and then probably only in harmless quantities. At other and normal times the coagulant is retained on top of the filter medium.

Coagulants have been used in both rapid and slow filters. In the former they are always essential; in the latter only when the water contains fine clay. After scraping the slow filters, a single dose of

coagulant may quickly re-establish the normal condition, and thus avoid the otherwise necessary waste of water for several days. Mr. Hering.

In the case of slow filters, Mr. Fuller attaches great importance to the difficulty of adding the proper amount of coagulants at the inlet to the settling basin, which difficulty is removed in the case of rapid filters by the possibility of readily changing the amount as the water enters upon them. This, however, is not a serious question. An examination of the water is, or should be, made at the inlet to the basin, as well as at the filter. Experience at the works will soon indicate the proper allowance of coagulant for every condition of the water, on those comparatively infrequent occasions, when a quick variation in the composition of the water takes place. In any case, this difficulty seems to be of less importance than securing the most perfect mingling of the slight quantity of coagulant with the large quantity of water to be affected.

Mr. Deacon prefers the slow filters because the quantity of coagulants, necessary in rapid filters, cannot be perfectly proportioned by an automatic apparatus. This disadvantage, he says, requires the employment of more intelligent men. Yet such a disadvantage is surely not serious, and may be measured by the cost element, which is generally the deciding one.

Regarding the duty performed by the sand grains, he adheres to the view that the sand is substantially but a support to the film on the surface of the bed, a view not originally held by the Massachusetts State Board of Health. By making the sand bed several feet thick we, therefore, merely add a factor of safety, so as to secure the desired action in the face of the natural irregularities found in the sand grains. This view is supported by Mr. Deacon, who calls attention to the fact observed by him that the water does not go down vertically, but in erratic lines, as a piece of paper falling through the air.

To obviate a number of the troubles with filters, Mr. Deacon uses automatic contrivances, not only to regulate the flow of water, as usually done, but also to immediately record and measure it on a diagram by a differentiating meter, a practice which is to be most highly commended as a means of knowing and of controlling the actual condition of the filter at any time.

Mr. Simin mentions the failures of slow filters in several of the larger Russian cities, and anticipates better results from the rapid process. These failures, in the writer's opinion, were probably due either to an impossibility of removing suspended clay particles of extreme fineness, or to bad management, or to both—causes which have appeared also elsewhere. Mr. Simin's eight reasons for preferring rapid to slow filtration, form, however, the most complete short presentation of the merits of the former which the writer has seen. While some of the reasons may yet be too sanguine, and while nothing

Mr. Hering. is said concerning the comparative demerits, they nevertheless indicate that a large field of profitable application for rapid filters may exist in some parts of Europe, as there does exist in some parts of America. The experiments with filters of both systems now being conducted side by side near Moscow will, it is hoped, throw some new light upon the subject from a European point of view.

Mr. Johnston makes a strong plea for the use of polarite instead of sand when filtering water, claiming that the degree of purification is higher and that the rate of filtration may therefore be more rapid. This substance has been known in America for ten years, and has been examined and tested. The chief disadvantage lies in the limited life of the material, so far as its purifying effect is concerned. When fresh it does better work than sand, but it does not permanently continue this ratio. Its greater first cost and the necessity of replacing it at intervals, would, in the long run, usually increase the entire cost of filtration above that of sand filters.

To overcome some of the difficulties met with in European slow filters, in getting pure as well as clear water at economical rates, the practice of double filtration was introduced, and is to-day practiced in Zurich, Schiedam, Bremen and Altona. In the last-named city this practice is not found to be of great advantage, and was not adopted for the adjoining new Hamburg filters. In Bremen it gives very satisfactory results. In Schiedam the advantage is questioned. The method of operation differs in each of these instances.

Professor Hilgard, in his discussion, describes the double filtration plant at Zurich, where the lake water is given a rapid preliminary filtration, before it is passed through the regular filters, for the purpose of removing chiefly the large number of minute organisms which had decreased the effectiveness of the single filters to about one-fourth of what it had been originally. The chief advantage of forcing air through the sand when washing the preliminary filters in Zurich should be sought in its mechanical and not in any biological effects. The bacterial results in Professor Hilgard's table confirm the belief that many bacteria get into the water after it has left the filter and before it is taken for use in the buildings—a matter which is often lost sight of in other cities, when pronouncing upon the work of filters.

When drinking water is highly charged with iron, Mr. Palmer advocates the use of polarite for preliminary and rapid filtration, prior to turning the water upon the ordinary filter bed. No details are given as to the durability of this material and the cost of the process, as compared with other means of removing iron. The experience gained with aeration seems to indicate that for equally good results the polarite could remove the iron as a permanent process only at a greater cost than the more common aerating process.

The removal of iron, the softening of water and the Zurich case,

substantiate the suggestion that economy will often result from a differentiation of the process, by a separate removal of each group of matter, similarly to what has been found advantageous in sewage purification. Mr. Hering.

Mr. Roechling believes that the methods of filtering drinking water and sewage are "totally different," "that the conditions of work in them are reversed." The writer does not understand the two processes quite in that way. In both cases we have water as the main substance, and in it are suspended or in solution: dead organic and mineral matter, living organic matter, such as bacteria or other minute self-reproducing organisms; and we have in both cases a loss of oxygen in the water. The difference in the cases of raw water and sewage is merely one of degree, and the process of purification varies with the amount of organic matter, its stability and the amount of oxygen present. In raw water selected for drinking there is sufficient oxygen to enable the bacteria to nitrify or convert the dead organic matter into mineral salts by the single process of oxidation; in sewage the oxygen present is not sufficient, and an intermediate process of putrefaction takes place, succeeded, however, by a process of oxidation, for which putrefaction is sometimes but a necessary preparation.

Dr. Kemna sees great advantages in the slow method of filtration, and recognizes it as the process of Nature when converting surface rainwater into spring water by percolation.

Dr. Rideal has indicated to us a new advantage in the use of alum by indirectly preventing plumbism. The sterilization of water by heat appears to him to be "the coming question." While it is not clear, to engineers, how the maintenance of a temperature sufficient to kill the pathogenic organisms "does not necessarily involve the expenditure of any energy," it may be well to call attention to what is the most economic method known to raise the temperature of water to the boiling point for a few moments at least. By a simple contrivance, arranged by Mr. John Forbes, two vertical currents of water are kept flowing past each other in contiguous passages, so that their heat is equalized by convection. The water, upon reaching the top of one column, is heated to the boiling point, and then, in descending, gradually imparts its heat to the water rising in the first column, and finally escapes with a temperature only 2 to 5° higher than the original temperature. The water is thus actually boiled, with a very small expenditure of heat, only enough to raise its temperature from 2 to 5 degrees.

Mr. Hunter does not place much weight upon bacterial analysis; yet, with all its shortcomings and uncertainties, such an analysis gives us to-day the best known index for the sanitary condition of the water, and has brought about the late development and true understanding of the essentials and non-essentials of filtration. Without it we would

Mr. Hering. have no measure of the sanitary value of a particular filter except the sick and death rates, which are affected also by other causes.

From the entire discussion it appears that no strong reasons have been brought out in favor of one and against the other of the two methods of water purification. Both methods have merits and a field of application, and their efficiency and cost do not differ very greatly in any case. In formulating their relative advantages, substantially as Mr. Fuller has done, we may say, that the slow method generally has advantages for waters which are non-clay bearing and low in color, which are cases where the rapid method is generally less efficient and greater in cost. On the other hand, the rapid method, with obligatory use of a coagulant, has advantages for waters highly charged with sub-microscopic particles of clay or which are highly colored, in both of which cases the slow method is less efficient.

As there is sometimes only a slight difference in the cost of the processes, or in the results of purification, the rapid method may sometimes be preferable solely because it requires a smaller investment of money at the outset, although a larger annual charge for operation, which in some cities, limited in their borrowing capacity, is an essential condition for the introduction of water purification works.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

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No. 886.

HEIGHT OF BUILDINGS.

An informal Discussion at the Annual Convention, London, England, July 2d, 1900.

Subjects for Discussion:

(1) "What Considerations Should Limit the Height of Buildings?"
(2) "Do Recent Developments in Construction, Sanitation, Intercommunication and Economy of Administration, warrant the Removal of all Restrictions?"

By Messrs. Corydon T. Purdy, Reginald Pelham Bolton, Robert W. Hunt, W. E. Riley, H. de B. Parsons, R. J. Gifford Read, Oscar J. Kirby, John F. Wallace, Maurice M. Sloan and William Copeland Furber.

Corydon T. Purdy, M. Am. Soc. C. E. (by letter).—High buildings are the outgrowth of commercial considerations. They facilitate business, they increase ground rentals, and they offer opportunity for profitable investment. Mr. Purdy.

As the American mind is not hampered by precedent or biased by ancient customs, these considerations have easily outweighed popular prejudice, and even small American cities aspire to high buildings. In Europe there is more conservatism, but all the world is now striving for commercial advantage, and the pressure to make buildings larger and higher will probably increase there as it has in America.

Mr. Purdy. The problem as to whether or not the removal of restrictions as to height is warranted must be faced. The question is pertinent, therefore, in both Europe and America.

It is the writer's purpose merely to outline for discussion this topic: "The Limitation of the Height of Buildings," and to mention some of the points of interest exhibited by the photographs. Plate XVII, Fig. 1, is a view of New York City, from the North River, well south toward the Battery. In less than a generation, high buildings have changed the sky line of this part of the city completely. A few years ago the regulation five-story building line was unbroken except by the towers of churches. At present, Trinity spire, for many years

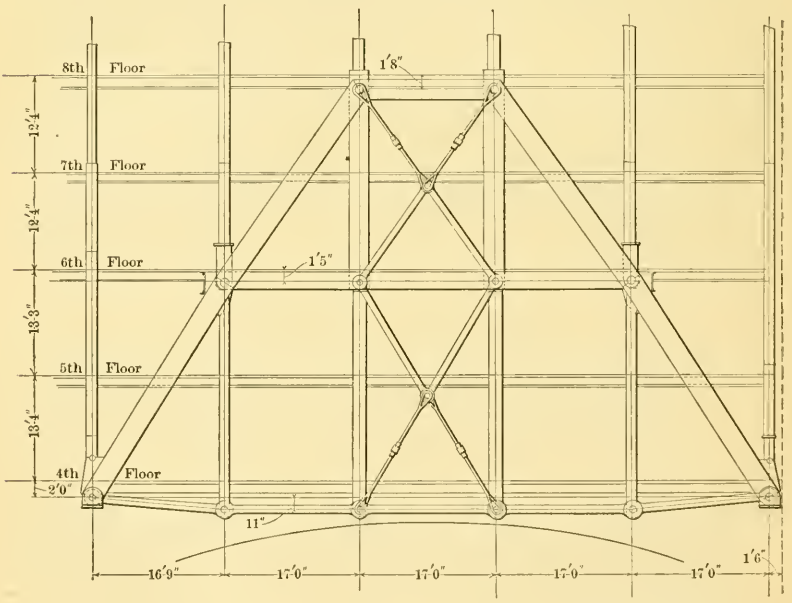

FIG. 1.

the highest structure in America, is looked down upon from the office windows of Wall Street and Broadway.

Many of the constructive problems in steel buildings are complicated in character, and they are not only so because of the variety of loads and strains which are sometimes involved, but because all lines of construction have to be worked out with reference to the uses of the building, the arrangement of its interior walls, and their openings.

The arrangement of the trusses over the great ballroom in the

FIG. 1.—SKY LINE OF LOWER NEW YORK CITY.

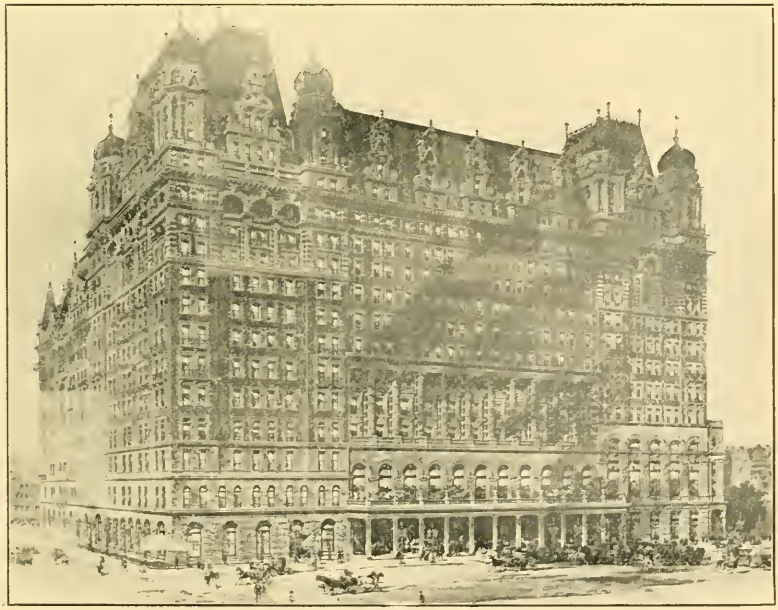

FIG. 2.—THE WALDORF-ASTORIA HOTEL, NEW YORK CITY.

Mr. Purdy.

Waldorf-Astoria Hotel in New York City, is a marked illustration. The lines of these trusses are shown in Fig. 1. The ballroom is 100 ft. long and 85 ft. wide and is without columns. It is three stories high. There are two great trusses sandwiched into the interior walls of the four stories directly over the ballroom, and their construction is worked out in such a way that their presence would never be suspected by people occupying the rooms immediately adjoining them. They carry 12 stories of bedrooms, weighing, with a reasonable allowance for live load, some 20 000 000 lbs. Some members of the trusses are heavier than can be found in any other structure in America. The lower chords are made up of nests of 12-in. I-bars carried on 12-in. pins. There is also one girder in this building 35 ft. long, which weighs 85 tons, and one column which carries 5 400 000 lbs.

The completed Waldorf-Astoria Hotel, Plate XVII, Fig. 2, is the largest and finest structure of its kind in America, and possibly in the world. It cost about $5 000 000.

The Old Colony Building, Plate XVIII, Fig. 1, is one of the notable structures in Chicago. While it is narrow and high, it is also one of the stiffest high buildings constructed in that city. The scheme for the stiffening or bracing consists of four vertical lines located equidistant from the axis of the building. The portals, shown in Fig. 2, are placed one on top of another in such a way that the shear is conveyed directly from one to the other, and the columns are practically incorporated with the portals as a part of them. Their chief advantage is in the opening they afford between rooms. For all practical purposes they do not divide the space at all, and yet the desired bracing is obtained.

The exterior walls of this building were in process of construction at different heights at the same time (Plate XVIII, Fig. 2). This cannot be done in buildings of massive construction. The new building for the North American Trust Company, Plate XIX, Fig. 2, recently completed on the corner of Cedar Street and Broadway, New York City, was built in the same way. Plate XIX, Fig. 1, shows the completed building and also affords an excellent illustration of the splendid light areas which can be put in buildings of this kind.

Some buildings are built too high for their width. That at the corner of Wall and Nassau Streets, in New York City, Plate XX, Fig. 1, might be mentioned as one of this class.

The highest building in America is that known as the Syndicate Building on Park Row, in New York City. It is shown on Plate XX, Fig. 2.

The Broadway Chambers, Plate XX, Fig. 3, is another of the new high buildings in New York City. This building is exhibited by models at the Paris Exposition, where every feature of its construction is illustrated.

Mr. Purdy

Where buildings are now limited, the limitation is due to one or more of four things:

(1) Legal enactment;

(2) Economic considerations;

(3) Conditions of construction, concerning which law may not specify; and

(4) Popular prejudice.

In New York City the only legal limitation to the height of fireproof buildings is a provision that the overturning moment due to wind pressure shall not exceed 75% of the moment of stability, with the wind pressure taken at 30 lbs. per square foot, over every square foot of surface exposed. In Chicago the legal restriction has recently been fixed at 135 ft.

There are many things to consider in either fixing or removing restrictions to the height of buildings by law. First of all there is popular prejudice. Then there are:

The disturbing effect on land values;

The exclusion of sunlight from the streets;

The congestion of traffic;

The increased difficulty of fire protection;

The overburdening of established water supplies;

The inadequacy of drainage and sewers already constructed;

The disturbing effects on the business of building trades, architects, engineers and contractors; and last, but not least,

Æsthetic considerations.

Some of these considerations are more important than others, but all bear upon the problem. The writer believes that the height of buildings should have a reasonable limitation, but it seems to him that most of the real objections to the so-called "sky scrapers" do not hold with much force against buildings of fourteen or fifteen stories, or less.

High buildings have become possible through the use of new materials, the new use of old materials, notably the use of steel, and the development of the elevator.

It is also worth noting that the new buildings are better and simpler than the old ones.

They have more light.

The interior arrangement may be made in any way which may be desired and that arrangement can be changed at any time if desired; while

The strength of the building is in the steel frame, and that strength is susceptible of accurate computation, which brings with it a simplification of all problems of construction.

In conclusion, the writer desires to call the attention of our members to the fact that though in America we have wonderfully

FIG. 1.—THE OLD COLONY BUILDING, CHICAGO

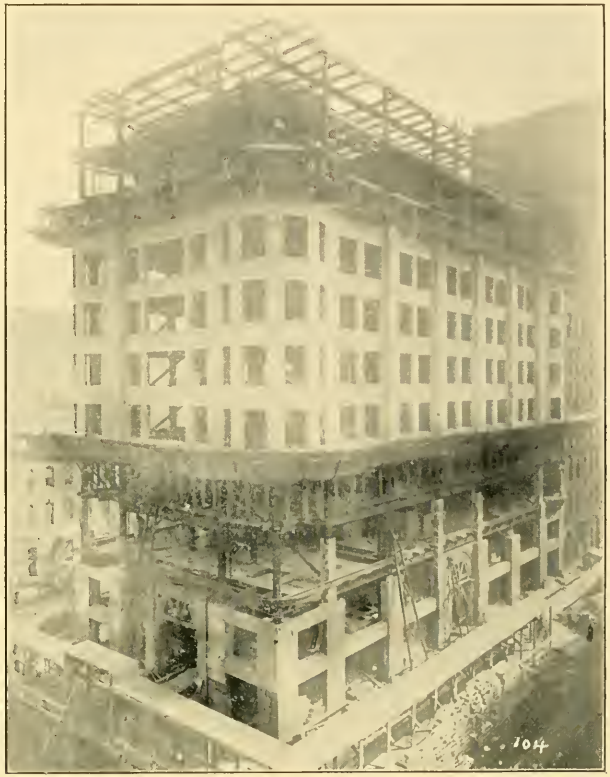

FIG. 2.—THE OLD COLONY BUILDING DURING CONSTRUCTION.

Mr. Purdy.

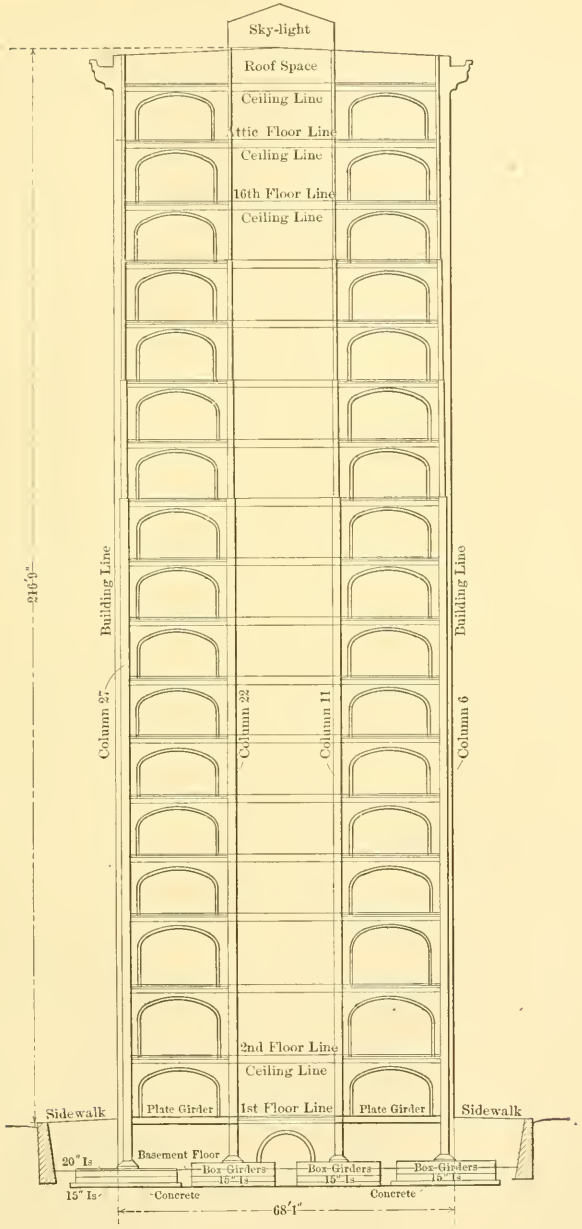

FIG. 2.

Mr. Purdy. developed the steel-constructed building, we owe a great debt to Europe for many of the ideas which have been used therein. It would be generally correct to say that European engineers have invented and Americans have developed.

One instance of this is sufficient to illustrate. As long ago as 1854, an asylum was constructed at York, England, by the Society of Friends, in which the floors were made of flat hollow terra cotta arches, of almost exactly the same pattern as we are now putting into our office buildings. They were designed by Mr. Pritchett, an Englishman. It has been repeatedly claimed by American writers that this arch was of American origin, but the first arch of this character was not constructed in America until 1873. Mr. Pritchett carried his arches over spans of 4 ft. 6 ins., which also accords with present practice. They were carried on cast-iron beams. The rolled beam had only been invented a year or two before.

Mr. Bolton. Reginald Pelham Bolton, M. Am. Soc. C. E. (by letter).—The writer's professional connection with the equipment of large office buildings and hotels in New York City leads him to the conclusion that the limitation of the height of buildings should be left to practical and economic action, and that legislation or municipal interference cannot produce desirable results.

The law of England, protective of the right of so-called "ancient light," has operated to prevent the development of buildings and their consequent improvement. Progressive increase in the value of land in business centers will operate to the disadvantage of tenants unless relief be given in the only possible direction, viz., upward.

Concentration of trades and of business in certain centers is desirable and economical, and the advantage of the community at large is advanced thereby. Therefore an inconvenience which may temporarily operate to the disadvantage of individuals should be endured if the general welfare is advanced.

The writer believes that these general propositions will meet endorsement, but, in this particular class of public improvements, it is clear that the apparent inconveniences are more imaginary than real.

It is true that in a country where the proportion of bright daylight is limited, the value of window light is not to be lightly estimated, but modern methods of artificial light have so advanced as to greatly discount the positive necessity of window light.

Further, in the construction of a higher class of buildings, in a city such as London, the primary steps are sure to be soon followed by a general increase. The "city" has been, within the writer's memory, practically reconstructed from a three and four-story basis, to a height of five to seven stories without general inconvenience, and the advantages of a single lofty structure would be so soon grasped

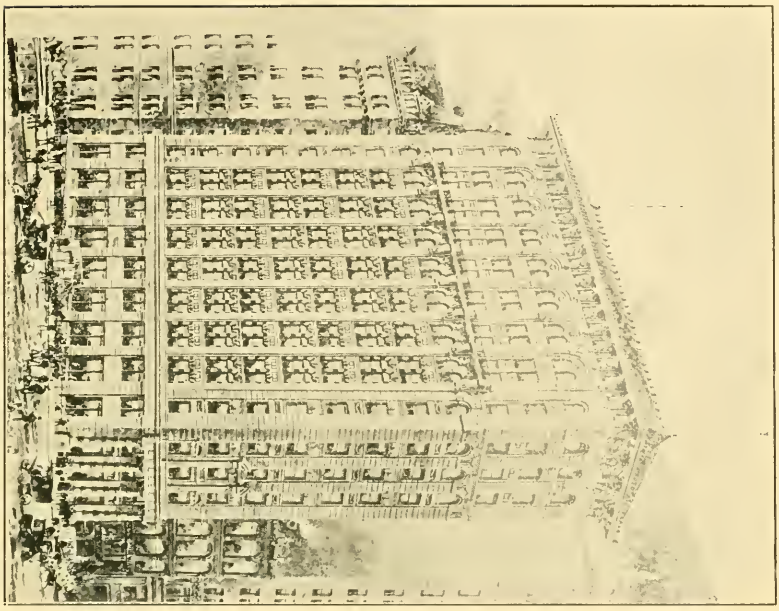

FIG. 1.—NORTH AMERICAN TRUST BUILDING, NEW YORK CITY.

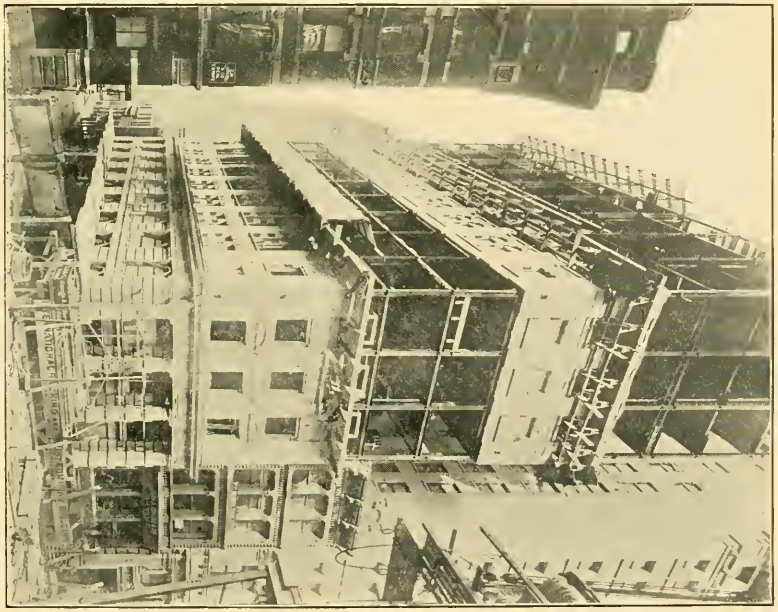

FIG. 2.—NORTH AMERICAN TRUST BUILDING DURING CONSTRUCTION.

that it would not be long before a general reconstruction of greater height took place. Mr. Bolton.

The construction of a tall building on a neighboring lot, while temporarily interfering with the immediate supply of light and, possibly, to some extent, of air, enhances the value of the neighboring properties, since on its success they become equally capable of increased development.

In the case of large office and hotel buildings the interior economy is such that artificial light and air supply can be and are generated at extremely low cost, less than is possible by public companies. Therefore a supply of both could be afforded to neighbors, in this way making up to them their temporary deprivations.

There cannot be any question that the general sanitation of a district is improved to an important extent by the construction of tall buildings, inferring, as this does, the effect of deep underground excavation and sub-soil drainage.

The writer can refer to instances in New York illustrative of this. When the Bowling Green Offices were constructed, the excavation extended 30 ft. below grade, and the sub-soil drainage was found to be extensive. Examination proved that a large part of it came from defective sewerage of the neighboring old buildings, the house sewer of one of which, a public restaurant, was found to be broken and choked.

This menace to health is, of course, now entirely removed by the sub-basement drainage system of the offices. Such cases could be multiplied indefinitely, and include wet walls and cellars dried, and insecure buildings supported and improved by neighboring structures of greater height and depth.

The relative increase in value of a piece of land, by reason of the construction of a lofty neighbor, may be gathered from what has proved to be the case in New York City.

Land, which with four-story buildings has barely paid expenses and taxation, has, with a lofty building, been able to earn all expenses; pay interest at the rate of 4% upon mortgages, the extent of which greatly exceeds the value of the original buildings; also pay a largely increased taxation, sometimes as much as five times the old rating; and, finally, leave a substantial profit on the net capital invested. The taxation borne by a certain property is relieved by the additional taxation placed upon an enlarged neighbor.

The writer believes that recent developments in construction, sanitation, intercommunication and economy of administration, warrant the removal of all restrictions, and further, he presents the proposition that natural causes will act as a restriction against excessive heights unless accompanied by proportionate areas. One of these is due to the limitations imposed by elevator service.

Mr. Bolton. The tall buildings which, in New York, have been the subject of most criticism, are chiefly those which, on a very confined area, rise to an extreme height. Their appearance, especially on the dead sides, is unsightly, and the architectural effects are almost ludicrous. The most flagrant example is the St. Paul Building, which has, perhaps, caused more criticism of the lofty building than any other.

It is in such buildings that the tenants of upper floors find their means of access of less convenience than elsewhere. For instance, in the building referred to, six elevators are installed, two serving the lower eight floors, two the intermediate, and two the upper eight floors. Each part of the building, therefore, is practically a portion of an eight-story structure, having only two elevators, a number certainly inadequate in comparison with others of equal height elsewhere. Moreover, the upper set of floors must be reached by a ride of 200 ft., equivalent to its location at that distance away from the main street.

The failure of architects to grasp the statistics of vertical traffic is not to be wondered at, the extraordinary feature being their attempt to deal with such a subject. They have not, as a class, learned that the necessary proportion of elevator mileage demands for a certain height a certain area, in which combination the proportions of a very high building becomes less offensive.

Further, the cost of maintenance of the upper floors in extremely high buildings is often disproportionate to their returns. These floors demand an excessive amount of heat for the winter season and increase the cost of house-water pumping. In addition, they must be debited with the larger proportion of the cost of the elevator service, and all these things being considered, they should produce a rental proportionately higher than floors below them. Such a proportionate increase is seldom applied, although some upper rooms are leased at high prices on account of the views they command, and their freedom from street noises. These features, and the elevation above the heated streets in summer, are found to be so well worth paying for, as to be the cause of the willingness of tenants to reside at an increased distance from the street.

The general conveniences of modern office and hotel buildings are so great as to justify much greater inconveniences than they cause.

The sanitary improvement over old and low buildings is great, and security from fire and increase of air space per tenant are worth general consideration.

It is due to the increase of size by means of height that mechanical improvements can be introduced which would otherwise be impossible except by the combination of a number of owners of contiguous properties.

In a recent article* the writer has given some of the figures of cost

* *Engineering Magazine*, June, 1900.

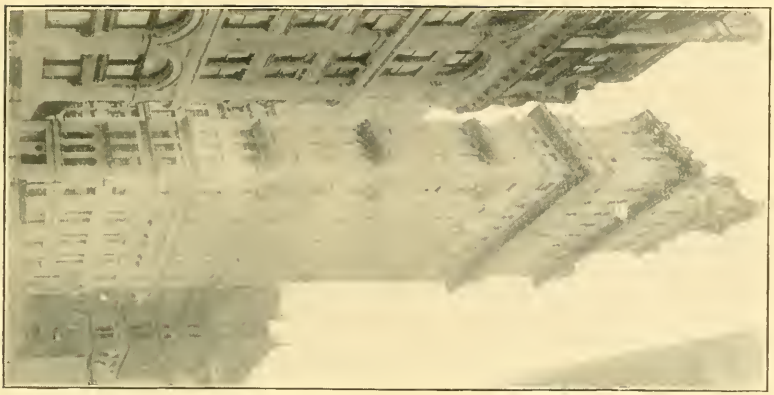

FIG. 1.—MANHATTAN TRUST BUILDING, NEW YORK CITY.

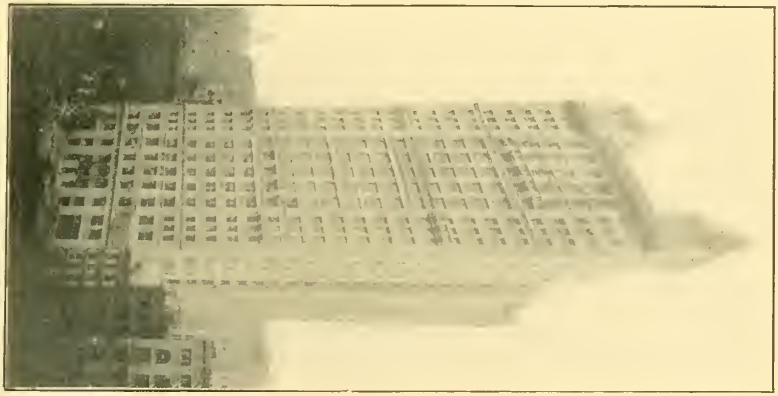

FIG. 2.—SYNDICATE BUILDING, NEW YORK CITY.

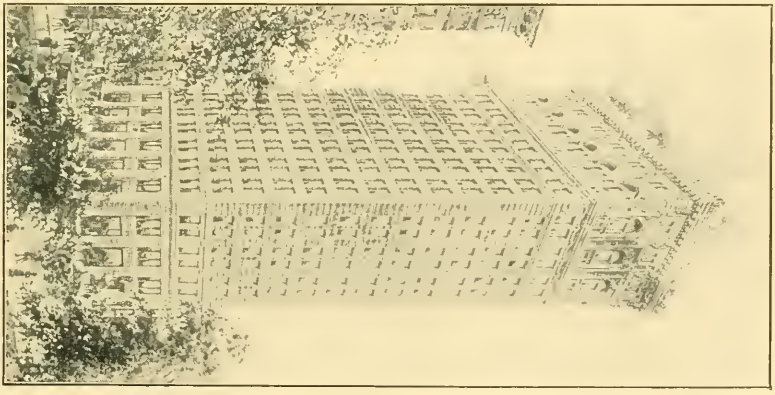

FIG. 3.—BROADWAY CHAMBERS, NEW YORK CITY.

of internal services, and has shown that the mechanical equipment reaches the proportions of a small central station, and can produce power in several forms under the most economical conditions. The fact that all the waste steam can be utilized for the necessary heating of the building in winter, and for other useful purposes such as heating water and cooking in summer, places these equipments in a position superior to the central station of public supplies. Mr. Bolton.

It is capable of proof that supplies of power in several forms can be sold by such large buildings to neighbors at figures very much less than the present cost of public supplies. Such advantages may thereby be presented to a contiguous property as will of themselves overcome the objection of interference with light or air, and the writer sees no reason why a tall building should not, by such arrangements, be a source of improvement to its older neighbors, and thus directly, as well as indirectly, add to their value.

Robert W. Hunt, M. Am. Soc. C. E.—Office rents vary in the different buildings and cities. The location controls the price just as it does under other circumstances. The speaker, ever since he has been in Chicago, has had offices in one of the first fire-proof high buildings built in that city. From the historical fact that the ground on which this building now stands was formerly occupied by such rattle-trap affairs that it was known, the city over, as the "Rookery," it was called "The Rookery" by the capitalists who secured the land from the city on a ground rent of 99 years. It is a building which will compare very favorably with any in the United States, although it was one of the first high office buildings. It is recognized from its location, history and associations as probably the most desirable building in Chicago, and as it can only hold a certain number of people, tenants have to pay quite the high figure named by Mr. Wallace, which is probably the highest rent charged by any of the buildings in the city. There are 3 200 people in the building. Its cost was about $1 800 000, and from its inception to the present time it has never paid its owners less than 12% on their investment, which makes it rather a profitable one for them. There are ten passenger elevators and one freight elevator, and the check which has been kept upon the number of people riding in them at different times shows that as many as 22 000 and 23 000 per day have been carried. Of course, that number does not represent different individuals, but the number of passengers carried. Every one living above the first story probably takes four rides a day, in addition to which there are business visits between the offices, and people coming to see the occupants. Mr. Hunt.

"The Rookery" is a town within itself. It has in the basement its own power plant, its own pumps, its own heating arrangements, its own electrical service. It has its own carpenters, its own painters, its own plumbers, and every mechanic necessary is employed in the build-

Mr. Hunt. ing permanently. They start from the roof and work down to the ground as regularly as the years roll on. By the time they reach the ground itself or the basement, the top of the building is in a condition to need their work once more, and they repeat the process, so that the building, after its 12 or 13 years of life, is just as good as it was the day it was built. The speaker is under the impression that the rate of rent his firm pays for New York offices in the Empire Building, at 71 Broadway, which is one of the newest buildings, is a little higher than that paid in Chicago.

Mr. Riley. W. E. Riley,* M. Inst. C. E.—The views of lower New York are hardly recognized by the speaker, to whom that part of the city was quite familiar twelve years ago. The geographical situation had always appeared to him to be a great difficulty; the narrow tongue of land between the broad East and Hudson Rivers, with the Harlem River on the north, completely hampers extension. The inventive mind of the American was driven to some expedient, and the evolution of high buildings was the result. The speaker desired, however, not to comment too much on American practice, but to describe European procedure.

This question had been approached from different standpoints. In the American tall building it is apprehended that the steel columns are the only portions of the structure which are carried to the foundations; they provide for the great resultant force of the building by carrying each story independently of the one above or below it. In the Building Acts in operation in London, Paris, Berlin and Vienna, it is laid down that the enclosing walls shall carry the structure, and, in the London Act of 1894, enclosing external walls are to be of the full thickness specified in the various schedules for at least half of their area.

The heights are also strictly limited in each of the cities named. In London, a building (not being a church or a chapel) must not be erected or subsequently increased to a greater height than 80 ft. (exclusive of two stories in the roof and of ornamental towers, turrets, or other architectural features or decorations), and, in a street laid out after August 7th, 1862, of less width than 50 ft., no new building shall be erected so that the height shall exceed the distance of the external wall from the opposite side of the street, without the consent of the London County Council. If the new street is over 50 ft. in width, the height may be carried to 80 ft., as if the building were in an old street; but if the street is, say, 49 ft. wide, the buildings upon it cannot exceed that height. The license allowed after 50 ft. width of street has been obtained may appear a little anomalous, but there is a very important further limit to height by providing for space in rear, and a governing angle.

The whole of the plot abutting on a new street, since 1894, must

*Superintending Architect of the London County Council.

have a space of not less than 10 ft. unbuilt upon, and from the extreme boundary a line of 2 to 1 (or 63.5°) must be drawn, and no projections above that line must be erected, except turrets, dormers and chimney stacks, and they must aggregate to limited portions of the width of the back frontage. Mr. Riley.

Some of the high buildings in New York must have very great cubical contents, but, under the London Building Act, the capacity is limited, *i. e.*, buildings for trade purposes, of a larger cubical extent than 250 000 cu. ft. and not exceeding 450 000 cu. ft., must not go beyond 60 ft. in height. That, again, is a very notable restriction. The greater cubical contents can only be considered on the ground that a particular trade renders it absolutely necessary for the operations of that trade.

Houses for the working classes cannot be erected to a height exceeding the distance of the front, or nearest external wall of a building, from the opposite side of any street, unless the working-class dwelling is 20 ft. from the center of the street. In Vienna, the height of dwelling houses to the top of the cornice must generally not exceed 82 ft., and the floor of the highest story must not be more than 66 ft. In Paris, the heights of buildings are determined by the width of the street upon which they abut, the height being measured from the pavement perpendicularly up the face of the wall, including all construction and ornamentation, and must not exceed 40 ft. for public ways less than 26 ft. wide; 50 ft. for public ways 26 to 32½ ft. wide; 60 ft. for public ways 32½ to 60 ft. wide, and a maximum of 66-ft. height for public ways of 66 ft. and over. No building must exceed seven stories above the ground floor.

During a congress at the Paris Exhibition which the speaker attended he noted that one of the Continental speakers drew some very forcible illustrations as to the height permitted for buildings in London, but the horror with which he spoke of buildings 22 stories in height, which are permitted in New York City, was quite indescribable. These are very decided comments on the proposition before us, viz.: "What considerations should limit the height of buildings?" These are the European views, which, as a rule, are not hampered by the geographical conditions which obtain in lower New York City, but the implied criticisms are clear where expansion is more practicable, as in other American cities. The conditions in London are such that it is very undesirable to permit a greater height than 80 ft. The width of the old streets is small, there are comparatively few buildings built of incombustible materials, and the difficulty of attacking and controlling a fire in a narrow street with a limited head of water needs no demonstration.

The limited sunshine in London should be regarded as a factor in considering the heights of buildings in relation to its open spaces.

Mr. Riley. Moreover, neighboring properties, which surely have vested interests in the light and air, should be duly considered. Lastly, the monumental character of some of the buildings in European cities, together with their association with the past history of their respective countries, would suffer more deplorably than they do even now if there were less consideration for the limitation of height. It appears clear, therefore, that it would be little short of a revolution in European cities to allow much further latitude in this respect.

The newest Building Code of New York City, which came into operation in October last, is an interesting document. In its provisions it is noticed that any hotel, lodging house, school, theater, jail, police station, hospital, asylum, institution for the care or treatment of persons, which exceeds 35 ft. in height, shall be built fire-proof, and every other building which exceeds 75 ft. shall be similarly constructed.

In a recent book * on fire-proof construction there are recorded some very interesting experiences on the behavior of high buildings in cases of fire. Illustrations are given of the Home Life Building, which the speaker did not remember to have seen, as it was probably erected since his last visit to New York. It is 192 ft. high, and was intended to be fire-proof, as all such high structures should be. A building of 5 stories adjoining it took fire and burnt out every floor of the Home Life Building above its own height. The disaster appears to have resulted through a shallow court 20 ft. wide, used for lighting the higher building, drawing up the flames and communicating them from one window to another in the progress of the fire in the higher building. The court acted like a purposely constructed chimney in its destructive effect, though how a reputed fire-proof building burnt so freely and fiercely is not so apparent.

A court of this character could not be built in a new building in London. The limit of height of the parapet above the sill of any such window, if used for a habitable room, is twice the root of area of the court, and, in case of a room used as an office, would be limited to the general height permissible under the act, but in no case exceeding 80 ft.

The destruction of the Windsor Hotel made a great impression in this country. It was substantially built, and it was stated in some papers to have been fire-resisting, though that statement is inaccurate, as it was elicited that the floors were of open wood joists; some of the partitions were formed with "quarters" and the internal finishings were wainscoting. Such a building would hardly be classed as fire-resisting here; at any rate, it burnt with such furious rapidity that it was practically gutted within an hour and a half. There is no doubt that every such building should be as uninflammable as modern

* "How to Build Fire-proof," by Francis C. Moore, New York, 1899.

science can make it. The responsibility of dealing with life and property in such a structure is a burden no public official should be called upon to undertake. In spite of the most heroic efforts, and an expert corps of fearless and brave men, many lives were lost and the property totally destroyed. In the case of the Home Life Building the water pressure was 108 ft., but that failed to save the building up to that height. Mr. Riley.

The means of escape in case of fire are constantly engaging attention in London, and, at the present time, further powers in regard to the provisions of the Factory Act are now being sought from Parliament.

With the foregoing examples on record he would be a bold man who would recommend any serious increase in the height of buildings in London.

The next question for discussion is, "Do recent developments in construction, sanitation, intercommunication and economy of administration warrant the removal of all restrictions?"

Modern construction has materially improved the conditions and lessened the risks of escape in case of fire, but construction has little beneficial effect upon provisions which deal with width of streets, proximity to other buildings, and the open spaces about buildings. It needs little demonstration to show that restrictions of height of buildings secure the essentials of light and air which are required for proper sanitation, intercommunication and economy of administration. There is no doubt that the ingenuity of American engineers has made the tall building work with a facility which could hardly have been thought practicable a few years back. The Produce Exchange is remembered as the highest building in New York fifteen years ago, and in London would still be considered as a very tall building, although it is now overshadowed. Expansion in London is inevitable, and efforts must be made to improve the communication with outlying areas. The solution appears to be looked for in building in the suburbs where the inhabitants can enjoy fresh air, and simultaneously provide rapid and reliable means of transport to the central points. There is no apparent demand for increasing the height of buildings, and none for the removal of all restrictions; here it would be a calamity to adopt any such legislation.

It is difficult to approach these questions of comparison without being dogmatic, especially when convictions are strong, but the few remarks submitted will, it is hoped, be accepted in another spirit. The subject is one of great interest, and the developments have been carefully watched. Many experts in New York City are bitterly opposed to high buildings, and they freely condemn the result of recent years' work as a hideous disfigurement of the city. These views find many supporters here, but perhaps a better acquaintance with the conditions which made the problem might soften the criticisms.

Mr. Riley. American engineers and architects are hampered by no insurmountable traditions. When a city railway was a necessity it was not put in an "underground" tunnel, as in London, but erected on stilts, and it answers its purpose very well. When a luxurious ship was wanted for the Fall River traffic, the *Pilgrim* (probably out of date by this time) was evolved. Her beam engines and other features appeared antiquated twelve years ago, but a closer acquaintance with the conditions revealed that the type adopted had much to recommend it. The systematic method of American thinkers has been to first grasp the difficulties to be overcome, and then attempt to solve the problem in the most practical and common-sense way, unbiased by prejudices. If the tall building is not all that can be desired, it is another effort to solve a difficulty, and, as experience is gained, it may ultimately lead to some satisfactory result, but it is not a suitable form of structure for this part of the world.

Mr. Parsons. H. de B. Parsons, M. Am. Soc. C. E.—Designing a tall building is a much more difficult problem than one would ordinarily conceive. "Fire-proof," "cage" and "skeleton" construction are three terms frequently used in connection with these buildings, which it would be well to define. By "skeleton" construction is meant that the outside walls of the building are self-sustaining, that is, they are carried up from the foundations as permanent walls. The interior of the building is supported upon a skeleton construction of steel or iron. By "cage" construction is meant that the building is supported *in toto* by the metal framework. The outside walls are mere curtain walls, and are supported themselves by the metal construction. Good examples of this construction have been shown by Mr. Purdy. The word "fire-proof" is used in a generic sense. It is impossible to conceive of any building which is intended to be inhabited and to contain the conveniences which the man of to-day demands, that could be built so as to be uninjured by fire. Fig. 1, Plate XXI, is a picture taken from the twentieth story of the American Surety Building, No. 100 Broadway, New York City, looking south, and gives a good impression of the height to which these structures tower. Wall Street, although only about 150 ft. distant from the building, is lost in the foreground. The Empire Building, No. 71 Broadway, which has been mentioned in this discussion, can be seen on the right in the picture. The low building shown on the left is that of the Union Trust Company, and was once considered tall, but it is now very much overshadowed by the next building beyond, namely, the Manhattan Life. The Bowling Green Building is seen in the distance and the Bay beyond. Looking down into the street below, one cannot refrain from imagining the difficulties which are encountered in the protection of these tall structures against the ravages of fire. Of all the elements which are tending to destroy these buildings, probably there is none

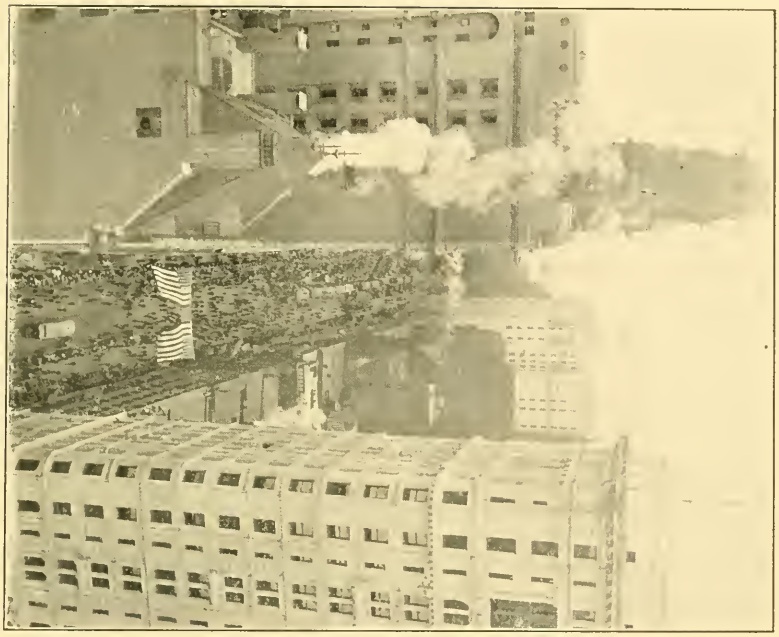

FIG. 1.—VIEW FROM TWENTIETH STORY OF AMERICAN SURETY BUILDING

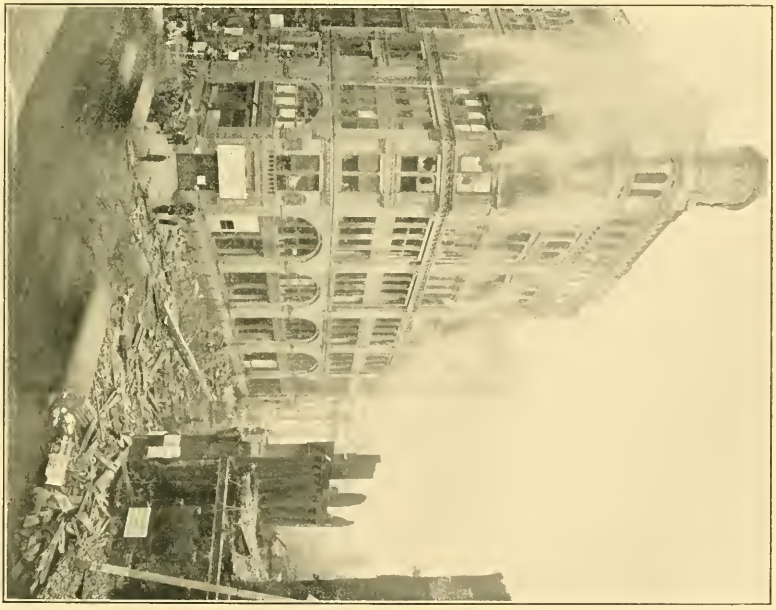

FIG. 2.—THE MANHATTAN SAVINGS INSTITUTION.

so potent as fire. It will be interesting to show a few examples, pick- Mr. Parsons.
ing out the worst cases rather than the best, in order to bring out the subject in the strongest light. In the first place, these buildings should only be constructed in the most fire-proof manner. It would not be good practice to build them according to the ordinary methods of construction, such methods as were mentioned in connection with the Windsor Hotel. Fig. 1, Plate XXII, represents the Windsor Hotel, New York City, which was destroyed by fire on March 17th, 1899. The hotel, of which this picture only shows a part of the front, was 200 ft. in length and was built in the form of a shallow U, the wings of which were 180 ft. in depth. In the rear of the hotel there was a court. The weakness of the building to resist fire was the design and the method of construction. At the center of the building and on the roof was located a large water tank. Through some error in the design, or through some change of plan, it is said that part of the building was supported from the roof by tie rods, the tie rods being carried by girders. The fire originated in the basement and spread with fearful rapidity throughout the building. It was known in New York as a dangerous building, owing to the wide corridors which ran from end to end on every floor, without any fire division walls whatever. This picture shows the building a few moments after the arrival of the engines. The fire was in the corridors behind the front rooms.

A few moments afterward, probably not more than two or three minutes, the photograph, Fig. 2, Plate XXII, was taken. A tongue of flame had burned away the flag, and the building began to tumble in a central vertical section. The break occurred so quickly that the firemen had hardly time to get out of the way.

Almost immediately after the last picture, the conditions were as represented in Fig. 3, Plate XXII. The tank had dropped from the top to the bottom, and that condition of affairs existed 35 minutes after the fire-alarm was given. Within a few moments afterward fire burst from almost every window simultaneously. Fig. 4, Plate XXII, represents the condition about 45 minutes after the alarm was sounded, and was taken just previous to the fall of the wall. It was only 5 or 10 minutes after the south wall fell, that the north wall gave way and the hotel was completely ruined.

Many imagine that buildings constructed of iron, steel, cement, glass, slate, and tiles are fire-proof and indestructible, but such is not the case. In the Washburn & Moen Company's Quinsigamon Mill, which was built on the "cage" construction plan, the walls were merely curtain walls. The window frames and sashes were all iron and steel, and the only wood used was in the floors and roof. The floors were built on the slow-burning principle. The structure was not overloaded at the time. It was a building used for the manufacture

Mr. Parsons. of steel springs, which are not in themselves combustible, and the reason the fire spread so rapidly was probably due to the fact that in one part of the building there was an oil-tempering room, and the vapors from the oil baths had condensed on the under side of the floors.* The columns yielded within from 15 to 25 minutes after the fire started. Experiments which were made by the Committee on Fire-Proofing Tests agreed very closely with the facts recorded in this fire. The plate to which reference is made illustrates very completely the destruction; and, when viewed from the standpoint of insurance, can it not be said with truth that iron and steel buildings will burn? Certainly, the destruction was complete.

The danger from fire in all large, tall buildings is greater from the hazard without than from any risk within. A fire, starting within a building which is of fire-proof construction, can be localized and put under control. But many buildings are subjected to an external hazard which is very great, owing to the large window areas and to the fact that there are windows on every floor, so that the fire may attack the building on all floors at once. Fig. 2, Plate XXI, shows the destruction of the Manhattan Savings Institution, New York City, during the winter of 1895 and 1896. It was at that time a modern building, and was well constructed so far as the materials were concerned. A building at the opposite side of the street caught fire; and the heat broke the glass of the windows, and was sufficient to warm the under-side of the girders which were exposed. They yielded; some, expanding, pushed out the walls, dropping the floor beams, and the building was a total wreck. Nothing was saved except some of the structure on the first or ground floor, and none of the fittings was in a condition to be used again.

On May 3d, 1897, there was a fire in Pittsburg, Pa., which originated in an old building containing a great mass of combustible material. Owing to the wind, it spread rapidly to a number of small buildings of similar construction, and attacked three modern buildings of the fire-proof kind. Reference may be made to the paper † by Corydon T. Purdy, M. Am. Soc. C. E., describing this fire. Fig. 2, Plate III, of that paper, shows part of the first floor of the Joseph Horne Dry Goods Store. It was a modern building, well constructed and well built, but in its construction there was in many points a total disregard of any fire hazard. The fire-proofing material stood well, and no fault could be found with it. The débris in the corner was caused by faulty design. On the roof there was a 5 000-gall. water tank, which was supported by unprotected iron girders and beams. Iron and steel, as shown in the Washburn Mill, will not stand unless protected. The

* Part of the ruins of this building are shown on Plate X, *Transactions*, Am. Soc. C. E., Vol. xxxix, June, 1898.

† *Transactions*, Am. Soc. C. E., Vol. xxxix, p. 121.

FIG. 1.—WINDSOR HOTEL FIRE.

FIG. 2.—WINDSOR HOTEL FIRE.

FIG. 3.—WINDSOR HOTEL FIRE.

FIG. 4.—WINDSOR HOTEL FIRE.

result was that the tank fell from the roof to the basement. The damage done to the structure by the falling pieces was probably greater than the damage done by the fire. The well, which extended up through the building to give certain architectural effects and to give light to the lower floor, proved to be very hazardous. The fire was drawn up by the current of air and carried to the other floors as though the well were a flue. Mr. Parsons.

Another view on the same floor of the same building is shown in Fig. 1, Plate IV, of Mr. Purdy's paper, and illustrates in a better manner the effect of the fire-proofing. All ornamentation is gone, but the fire-proof blocks were well preserved. Again referring to Mr. Purdy's paper, Fig. 1, Plate III, and Figs. 1 and 2, Plate V, show the exterior of the buildings after the fire. The damage in the Horne Office Building was very much less than in the Dry Goods Building, and was due to a better arrangement of fire-proof partitions. The location seems to be unfortunate, as the Dry Goods Store was rebuilt and has been burned a second time.

Mr. W. E. Riley has mentioned the Home Life Building, in New York City, as having been attacked by fire. Fig. 1, Plate XXIII, shows this building before the fire. It is 16 stories high, 192 ft. to the roof, and 242 ft. to the top of the tower. The low building at the right was of ordinary construction, occupied as a clothing house, and caught fire. The tall building next to the Home Life Building is the Postal Telegraph Building. The wind was blowing at hurricane force, unfortunately, from the northeast, so that it blew against the side of the Home Life Building, which contained a court or light well. The flames were drawn into the well, which acted as a chimney, and, as all the windows were unprotected by shutters, the fire entered simultaneously every floor of the building above the ninth.

Fig. 2, Plate XXII, shows the building after the fire, and also the well or court just referred to. Of course, the small corner building was completely ruined. The destruction of the façade was complete from the ninth floor upward. Below that level, the building was practically intact and was used again by the tenants within a very short time after the fire. The contents of the seventh and eighth floors were badly damaged by water. The elevators and the staircase were located next to the light well, so that the flames on entering the building had free access to every floor.

Fig. 1, Plate XXIV, is typical of any room in the Home Life Building, above the ninth floor, after the fire. The fire-proofing stood the test, and was found to be in such good condition that most of it was used over again without any expense. The partition walls had large transom windows which were unprotected, and were fitted with ordinary glass set in wooden sashes. When the fire entered a room, it simply broke the glass and passed on to the next. Had these partitions been of a

Mr. Parsons. fire-stop character, all damage would probably have been confined to the rooms next to the building which originally caught fire.

Fig. 2, Plate XXIV, shows a wire-lathe and plaster partition which had been built on the wooden floor. The floor was laid on wooden sleepers, leaving an air space between the fire-proof arches and the wooden flooring. If this building had been constructed, as was originally intended, and as all such buildings should be, by filling this air space with ashes or concrete, and by resting all partitions directly on the arches, the probabilities are that the fire would not have damaged the partitions. As it was, when the wooden floor had been burned away, the partitions had no support and simply dropped.

The greatest damage to the steel frame-work is shown in Fig. 3, Plate XXIV, which represents a view of the front room on the sixteenth floor. The large girder had its lower chord well protected, but the upper chord projected above the floor level. This girder was covered with a flooring, raised in a sort of step toward the front wall. The wooden floor soon burned away, leaving the upper chord entirely exposed. It became red hot and buckled, as shown. The photograph does not illustrate very well the damage to the façade and window trim, owing to the strong light. The missing fire-proofing blocks were removed from the column after the fire for the purpose of inspection, and this column, as well as all the others, was found to be in good condition. The blocks seen on the floor were knocked off by the firemen and by the City Building Department, and were not displaced by the fire itself. The heat in this room was very great, and, as can be seen, no plaster remained on the walls.

The marble front of the building, from the ninth story upward, was taken down and rebuilt. The Postal Telegraph Building was injured on the upper story by fire and only damaged by water on the floors below.

As a fire-stop, these tall buildings certainly acted marvelously well. It would be frightful to contemplate what might have happened during that gale of wind had these buildings not been there; but, of course, tall buildings are not intended as fire-stops.

In case of fire, any door or window opening is dangerous. The way to stop a fire from spreading is to confine it to the place where it originated. If circumstances necessitate a large floor area, then make such area a fire-proof floor and ceiling, cutting off all communication with the rest of the building. If possible, put the staircase in a tower-like structure on the outside, and give access to each floor by a bridge. Make all communication from one floor to another pass across the bridge, up or down the stairs, and back over another bridge to the next floor. It is best to subdivide large areas by fire-walls or by fire-proof partitions and have as few openings as possible. It is an impossibility not to have doors and windows in tall buildings, but such openings

FIG. 1.—THE HOME LIFE BUILDING.

FIG. 2.—THE HOME LIFE BUILDING, AFTER THE FIRE.

create a hazard. When the divisional walls are cut in numerous places, it is hard to conceive of a fire-proof building which could not be injured by fire. The danger from without is usually far greater than that from within. Mr. Parsons.

In the United States, many prefer the "slow-burning" construction, and a wooden building made on this principle will outlast an ordinary steel building, unless the steel building is put up in the most approved manner, with the members covered with the best fire-proofing, but this construction is not suitable for very high buildings. The "slow-burning" method simply consists in using large pieces of timber for posts and girders, so that the fire can char to a depth of 2 ins. or more without impairing the strength necessary to support the loads, and constructing the building so as to leave no air spaces. It takes a long time for fire to char a large stick to that depth, and by that time the fire department can come to the rescue.

The lower floors of the Home Life Building were uninjured, because they were protected by the brick wall of the adjacent building which first caught fire. There is nothing better than an ordinary brick and cement wall, and such a wall protected the Home Life Building. The flames had to pass over its top, and then had an upward tendency from the draught of the well and elevator shaft, and the first windows which really got the attack of fire were on the level of the ninth floor. The ninth floor practically marks the limit of protection from the Fire Department in the street.

Tall buildings are now constructed with stand-pipes, which are large pipes 6 or 7 ins. in diameter, extending from the street level to the roof. At each floor there are arrangements whereby the fire hose of the building can be connected to the stand-pipe, and the City Fire Department engines connect with the bottom of the stand-pipe. The engines pump water into the stand-pipe and the firemen use the hose which is on each floor. In that way they save much valuable time as they do not have to carry up their own hose to great heights. In some tall buildings the elevators or lifts are in commission night and day for the purpose of fire protection, and in case of alarm the elevator is immediately sent to the lower floor to be ready to carry the hose or the firemen to the proper point.

The fire-door which has generally been found the most serviceable in the United States is that known as the Underwriter's door. It is simply a wooden door sheathed with tin. The door is often strengthened by being made of double plank laid diagonally. The coating is of ordinary tin well lapped. These doors will stand an enormous amount of heat before they fail. They are carried on a traveler overhead. The traveller is on an incline, and during working hours the doors are held back by a cord, a wire or a chain, in which there is a fusible link. The link operates the same as the sprinkler; at a certain

Mr. Parsons. temperature, which is quite low, it melts, and the door closes automatically. Many forms of patented fire doors have been used, but none with such universal success as the Underwriter's door.

Mr. Read. R. J. Gifford Read, Assoc. M. Inst. C. E.—Some wonderful examples of the disastrous effects of fire on ironwork have been shown in this discussion, and attention specially called to the efficient protection which the fireproofing afforded. The speaker would be glad to know what material the fire-proofing was made of, whether it was of the nature of plaster, terra-cotta or brick.

He thinks that the views show in a very clear and impressive manner the effects of fire on these high buildings.

With regard to the case where the fire originated in the eighth story and spread upward through the elevator shaft, he would like to ask how the people who were above escaped, and if, in tall building construction generally, there is any other method provided for escape than by the elevators.

In certain high buildings in London on which he had been engaged, provision for escape was made by outside balconies and staircases, or by outside galleries connecting one building with the next, or by escape ladders to the neighboring roofs, but these would scarcely apply in the very high American buildings.

Mr. Kirby. Oscar J. Kirby, Esq.*—In high buildings of six and seven stories the speaker uses corrugated steel doors with asbestos sheeting on either side, so as to cut off entirely one room from another, the grooves or corrugations being filled with ground clinker to give solidity to the doors. Bolts or rivets keep the faces of the doors compactly in position, the bolts being so arranged, where necessary, that they can be covered over with moulded asbestos to give the faces of the doors the appearance of wrought woodwork.

These doors, unlike the composite fire-proof tin-covered wooden doors, do not warp in a fire and break loose from the hinges or fastenings.

Mr. Wallace. John F. Wallace, President, Am. Soc. C. E.—The cost of high buildings in Chicago has varied from about 30 to 40 cents per cubic foot, and the rental for office room above the ground floor has varied from $2 to $2.50 per square foot per annum. Of course, this cost of construction and rental received varies largely between buildings differently located. The rental generally includes janitor's service, elevator service, and sometimes heat and sometimes light. These figures may have been modified in the last two or three years.

High buildings had, not their origin, but their highest development in Chicago, and they were not constructed in order to get something larger than existed anywhere else in the country, as some people might

* Engineer and Surveyor to the Batley Corporation.

FIG. 1.—TYPICAL ROOM IN HOME LIFE BUILDING AFTER THE FIRE.

FIG. 2.—PARTITION OF WIRE-LATH AND PLASTER.

FIG. 3.—FRONT ROOM, SIXTEENTH FLOOR, HOME LIFE BUILDING.

have an idea, but from the circumstance that the business portion of Chicago is confined to a very small area. The Chicago River comes in from Lake Michigan for a short distance and then branches in a northwesterly and southwesterly direction, forming a Y. In the district south of the river, and between the river and the lake, what is known as the south side, the principal business of Chicago is carried on. The railway stations all have their termini within a comparatively small area, and the wholesale as well as all the retail houses are located in this district. The price of land per square foot, therefore, became very high on account of the congested business, and it was impossible for land owners to obtain for low buildings a rental which gave a reasonable return. In order to overcome this, and in order to receive an adequate return upon the assessed and market value of the land, they were forced to construct high buildings. In New York and other large cities of the same character it has not been a question of architectural beauty, nor a question of sanitation, but simply a business and commercial necessity. Therefore it may be interesting to our English brethren to have some idea of what those buildings cost and what rental they return. This would seem to be the key-note of the whole question. Mr. Wallace.

In order that our English brethren may understand the situation in America it may be desirable to say that all American engineers do not favor the continuation of the construction of high buildings, even in large cities, and are very much divided in opinion on that subject.

Answering Mr. R. J. Gifford Read's questions: In the first case this fire-proofing is generally terra-cotta work. In the cage construction, in between the floor beams the floor is constructed of a series of terra-cotta blocks which are made with inclined joints and so formed that they make a flat arch. In order to make the blocks light they are made hollow; hollow tiling or terra-cotta work is also built around and envelops the columns, and in some cases it is joined by cement or is keyed around the columns. In the later constructions the fire-proofing is very carefully put around all the iron and the steel columns. Steel floor beams are not now looked upon as a fire-proof material. They simply furnish the ribs or the skeleton, as it might be called, on which the fire-proof construction is placed. At an earlier date concrete was used for these floors, and thin strips of wood were placed in it and the wooden floors nailed to these strips. In the later construction the floors are made of concrete on the top of this fire-proof terra-cotta work, and then small stones are put down in mosaic work and are then smoothed over. This makes the fire-proof floor, so that now, in a great many buildings, there is no wood except simply the doors and the window cases.

With reference to fire-escapes, in all American cities where high buildings are used, a certain number of fire escapes are now obligatory

Mr. Wallace. by city ordinances, and they are required to be constructed on the outside of the buildings, generally in connection with balconies and at the ends of the halls, and the number placed is in proportion to the size of the building. They are inspected periodically by the city authorities. The fire-proofing of buildings has received of late years serious consideration. When these buildings were first constructed it was supposed that the iron and steel work was fire-proof and would not yield so readily to the heat. The character of the occupancy of the buildings has a great deal to do with them. They are well adapted to office buildings in the congested portions of the city, but they are not adapted to retail business or dry good stores, or to any business where the contents are highly inflammable, except where they are constructed with fire walls and fire partitions and with isolated rooms. In Pittsburg, Pa., in a hotel of recent fire-proof construction a fire occurred in a suite of rooms, and two rooms with their furniture were entirely destroyed and burnt out. The occupant of the adjoining room did not know there was a fire until informed of it the next morning. That was due to the careful design of the building in reference to fire walls, terra-cotta work and the use of cement, concrete and mosaic floors, and the absence of transoms over the doors.

Mr. Parsons has given us some information regarding fire doors, and the speaker can answer with regard to some warehouses in the City of New Orleans, which have been constructed under his immediate control. One warehouse was 1 200 ft. long and 100 ft. wide, for the storage of cotton, cotton-seed oil, and products of that kind, which are highly inflammable. That building was constructed with brick foundations up to the level of the floor and then filled in with earth, upon which a concrete floor was constructed, dividing it up into sections. Very narrow openings were left between the different sections, and were protected from each other by what might be called twin doors, with an air space between them. It is not permitted to pile any material within a certain distance of these doors.

Mr. Sloan. Maurice M. Sloan, Esq. (by letter).—The public good, or the common welfare of the community, is the only consideration that should regulate the height of buildings, and the law of commonweal, though sometimes slow, is ever present and a governor of all things human.

This is undoubtedly the legal aspect of the discussion, for it places the rights of the owner, on the one hand, to build as he chooses, over and against the rights of the people, on the other, to restrict as they deem advisable.

The restrictions placed upon the height of buildings by a community should be confined entirely to such as they have a right to impose. That public prejudice and æsthetic considerations should interfere with the owner's individual right to build as high as he pleases is unreasonable and absurd. Public prejudice is undoubtedly

a strong factor, but the world would have remained in an exceedingly primitive state if it had always obeyed the admonitions of this fickle and somewhat unreasonable goddess, while æstheticism, with its principles and doctrines, so often results in mere whimsicality or grotesqueness that it cannot be taken as an imitable law, and is not worthy of consideration in so vital a question as that under discussion. Mr. Sloan.

Where the height of a building, however, affects the safety of either the individual or the community, and jeopardizes life and health, or even stagnates commercial operations by congesting the traffic in the street upon which it is located, restrictions should be fixed, and too much care cannot be exercised in determining what should be the limitation.

That a building may be constructed to any height with perfect safety is a proposition generally accepted, though many conjectures arise as to the terrible havoc and loss of life that would be caused should earthquakes of great severity, or cyclonic storms of unusual violence, devastate the modern city of skyscrapers. These conditions are, however, of such a cataclysmic nature as to entirely preclude them from an argument for or against high buildings.

From a hygienic point of view there is little doubt that tall buildings are a menace to the health and happiness of a community. Sunlight is a natural disinfectant, and should be allowed to penetrate to every city street, flooding it at least for a few hours each day with its purifying rays. The fulfillment of this seemingly unimportant condition is really of vital moment, and can illy afford to be neglected in a great city teeming with a population susceptible to decimation by contagious and infectious diseases.

It would almost seem that this consideration was of sufficient importance alone to regulate the height of buildings, and would give a basis to work from, at least in temperate climates, that would insure light and airy streets, free from the miasma which is so typical of the narrow passageways between high buildings in a city. Such a basis of determination would not only satisfy the exacting requirements of the sanitary engineer, but would in many cases meet the demands of the æsthetician.

In considering the necessity for providing sunny and healthful streets, take for example the conditions as they exist in New York. This city is in latitude 40° 43′ N., and the sun's declination is 23° S. in midwinter, that is, on the 22d of December. The altitude of the sun at noon is found by the formula:

$$\text{Alt.} = 90° - (\text{Lat.} + \text{Decl.}),$$

and equals 26° 17′, giving a zenith distance of 63° 43′. Assuming a building 100 ft. high, located on a street extending east and west and letting the width of the street represent the tangent, and the height of the building the radius of the angle 63° 43′, the width of the street

Mr. Sloan. necessary in order that the sunlight may penetrate to the foot of the opposite side walls would be 202 ft. Such a street would be entirely too wide and pleasant for the economist and utilitarian, but, after all, would it not make a city of such healthfulness and beauty as to be the envy of the world.

While it is true that the sunlight could never, at least in winter, penetrate to the entire street, the reflected light from the opposite wall would be all that was necessary to promote a healthful atmosphere and light buildings and streets. The width of the street, figured on this basis, would also take care of any possible traffic brought about by the usual commercial transactions. It is universally understood, and conclusively proven, that recent developments in sanitation, intercommunication and economy of administration, remove all restrictions relative to the height of buildings. There are, however, limitations imposed by economical construction which will prevent the height of buildings from exceeding that already attained at the present time.

One of the factors which will tend to limit the height of buildings is due to the fact that as the distance from the foundation to a story increases, the investment required to construct that story is likewise increased, and as the returns on the investment are not subject to such an increase, there is a limit beyond which it is unprofitable to go. Such a condition arises from the fact that the increased sectional area required in the columns to support a given floor extends not only throughout the length of the columns in the tier below, but throughout the entire distance from the floor in question to the foundations. Similar considerations apply as well to the wind bracing, especially where diagonal bracing is used, and where the building is regarded as a cantilever beam supported at the foundation, for the wind bracing at any floor must be proportioned to sustain the increment of stress from all the floors above.

It is such conditions as these that have made the growth of the ocean liner and the locomotive slow, and kept the span of bridges within reason, and will likewise prevent the design and construction of buildings of greater height than those at present erected.

Mr. Furber. William Copeland Furber, M. Am. Soc. C. E. (by letter).—The real trouble with the high building and its relation to adjacent buildings is, that it has become a misfit in the present city block plans and streets, and, because of this misfit, this relation has become one of the principal factors in determining its height, sanitation, and intercommunication. Many of the objections to the height of buildings disappear as soon as they are isolated from their neighbors a sufficient distance to allow circulation of air, an adequate amount of light to answer sanitary requirements and the needs of tenants and neighbors.

The high building is a great concentration of available rentable

space for the transaction of business, with the least amount of waste time and energy. Its great aggregation of floor space within convenient limits, makes it possible to provide for all the proper conveniences and many of the luxuries which lighten labor, at a cost which justifies their purchase by tenants. Taking an average of say 12 ft. from floor to floor, every 12 ft. in height doubles the area of the ground occupied, so that, from a given point on the pavement, within a comparatively small vertical distance (which requires no laborious effort to reach), the available area of the land is increased as many fold as the number of stories. Mr. Furber.

With the acquisition of any advantage, however, experience shows that many drawbacks usually accompany it, but wisdom points the way toward a compromise of some of the good features of the one, to avoid some of the evils of the other. We cannot have all the land occupied, and the high building at the same time, without bringing in a train of evils, due to overcrowding. High buildings, like great men, need room to develop their capacities. The high building, to be suited to its surroundings, demands a rearrangement of the city block plans, making each large structure a block of itself, with wide main streets and subordinate streets of sufficient width for fire protection, light and ventilation. The high building should have no party walls, and the necessity for them would disappear upon making each structure a unit. Congestion of street traffic would also be relieved to a great extent by the increased area of street room thereby available, and the right of "ancient light," which should never be ignored, would receive more consideration. The fire hazard, due to adjoining buildings, would be greatly lessened, and by the use of lines of distributing water pipes at the several story lines, known as the "Water Curtain," and fire shutters, with fusible plugs at points of greatest exposure, the danger of destruction by fire would be almost entirely obviated.

Such a radical departure from present block plans would require a readjustment of property lines and street lines, but it would soon justify itself by the advantages it brought with it. It seems hardly like intelligent action to make dark cañons of the streets, nor would it impress a student of the requirements as at all necessary.

If some legislative regulation required a minimum amount of sunlight on the streets, somewhat on the lines suggested by Mr. Sloan, coupled with the requirement that each building should stand detached, there would be not only a great gain for sanitation, cheerfulness and beauty of the street itself, but a great gain in the architectural treatment of the buildings, and a much-needed improvement in the sky line of the streets. Then, buildings designed under dissimilar ideas and requirements would not adjoin, and the sense of harmony, now so sadly lacking when two or more great buildings stand shoulder to shoulder, could not be ruthlessly disregarded.

Mr. Furber. The rearrangement of streets in Paris offers a precedent for intelligent action.

Æsthetic considerations alone should place no limit on the height of buildings, but the practical limitations of sunlight and air should keep this height within bounds of conservatism. The architectural design should be influenced by structural necessities, and correct æsthetic design should recognize the skeleton in the building and not attempt to conceal it. The root of the trouble is in attempting to treat architecturally a material of high resistance, like iron, with the same proportion—or slight modification of it—as stone. The fundamental basis of all architectural forms was originally the resistance of the material used to change of form from external forces, and the necessary dimensions for these forms guided the mind in shaping them. The age of stone architecture is past, and it is folly to treat the high building in a style adapted to stone only. The iron structure demands a freer hand, and an acknowledgment that the material used possesses both tensile and compressive strength; and with this recognition of the material should come an acknowledgment, also, of its weaknesses in high temperatures, by providing sufficient covering for efficient fireproofing.

When the change was made from building with masonry piers to building with iron columns, the reduction in the area of the piers was so great and so easy, and the window space, or open space, seemed so desirable, that the size of the pier was frequently diminished to little more than the area of the metal column, with a minimum of covering. This has made iron buildings in many cases unsafe and dangerous to their surroundings, and has placed too great a responsibility upon an insignificant amount of fire proofing, and this defect alone has called for a limit to the height of structures which are so deficient in this respect.

Another defect in a great deal of high building construction, is the carelessness, or ignorance, of the designers in failing to take precautions to prevent corrosion, and when the first building begins to fail from the corrosion of the main lines of its support, there will be most likely a shrinkage in the estimated value of many other buildings built on similar lines, and the good buildings will suffer with the bad, because few will be able to tell one from the other.

There is little doubt that ironwork can be made practically rust proof, as well as fire resisting—within practical limits—if the proper thought and study are given to the problem, but because it is out of sight it is out of the mind of many who are intrusted with great responsibilities in this respect.

So, therefore, it seems as if the limitations of high buildings are practical ones, which should be squarely met, and the various factors making up these limitations must either be given adequate consideration, or a surrender made when terms cannot be honorably obtained.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

No. 887.

RECENT PRACTICE IN RAILS.

An Informal Discussion at the Annual Convention, London, England, July 3d, 1900.

SUBJECTS FOR DISCUSSION.

"The Progressive Increase in Weight; the Increase in Hardness, particularly in Carbon; the Sections in most General Use; the effect of changes in Weight, Composition and Section."

By Messrs. ROBERT W. HUNT, ALBERT LADD COLBY, WILLIAM R. WEBSTER, JOHN F. WALLACE, SIR LOWTHIAN BELL and J. D. SMELT.

ROBERT W. HUNT, M. Am. Soc. C. E.—As America came to Great Britain for the rails used on her first railroads, it is perhaps fitting that this discussion should be presented to the Society in London, and at its first English meeting. Mr. Hunt.

The facts relating to the introduction of rails into the United States, and, later, their manufacture in that country, are historically recorded, and will only be briefly referred to herein. In obtaining the data, free use has been made of James M. Swank's valuable work, "Iron in All Ages."

The first charter for a railroad in the United States was granted by the Legislature of New Jersey in 1815, but the road was never built. In April, 1823, the New York Legislature granted a charter to the Delaware and Hudson Canal Company to construct a canal and railroad for

Mr. Hunt. the transportation of coal from the anthracite coal fields of Pennsylvania to the Hudson River. The road was 16 miles long, but was not completed until 1829. It was on this road that the first locomotive was run in America.

On Saturday, August 8th, 1829, the "Stourbridge Lion," built in England, and weighing 6 tons, made its first trip; but its active life was short, as it was found to be too heavy for the superstructure of the road.

The first American-built engine was operated on the Baltimore and Ohio Railroad, in August, 1830. It was named "Tom Thumb," and was designed and constructed by Peter Cooper, who was somewhat restricted as to materials, as he had to use gun barrels for tubes. The whole machine weighed but 1 ton, and burned anthracite coal. The experiment was successful, and, based on it, later machines were designed and built; the first one of practical size and use being the "Best Friend of Charleston," which was constructed at the West Point Foundry, in New York City, for the Charleston and Hamburg Railroad of South Carolina. It went into active and successful use on that road in December, 1830.

Only some five years intervened between the opening of the first railroad in the world intended for general freight and passenger service—the Stockton and Darlington—September, 1825; and that of the first one in the United States for the same purposes—the Baltimore and Ohio. Its construction was begun on July 4th, 1828, and "cars were put upon it for the accommodation of the officers and to gratify the curious by a ride," in 1829; but it was not formally opened for travel until May 24th, 1830. It was then 13 miles long, extending from Baltimore to Ellicott's Mills, Md.

Thus, in railroad building, as in many other things, the Americans were early disposed to follow closely after their English relatives; and perhaps, like other younger people, were soon not satisfied to follow, but aspired to lead. Thus, the next passenger railroad to be constructed was the Charleston and Hamburg, already mentioned. This was opened for public use in December, 1830. In September, 1833, it was completed for a distance of 135 miles, and was "the longest continuous line of railroad in the world."

The nucleus, from which later came the great New York Central and Hudson River Railroad System, was the Mohawk and Hudson Railroad, chartered by the New York Legislature in 1826, but not begun until 1830, and opened for travel in 1831. It extended from Albany to Schenectady, 17 miles.

There were some earlier railroads, or more properly tramways, built in the United States, but those named were the first really commercial roads.

The rails used on these roads were of wood with flat bar-iron nailed

to their upper surface. The track of the Baltimore and Ohio Railroad is described as consisting of— Mr. Hunt.

"Cedar cross-pieces, and of string-pieces of yellow pine from 12 to 24 ft. long and 6 ins. square, and slightly beveled at the top of the upper side for the flange of the wheels, which at that time was on the outside. On these string-pieces iron rails were placed, and securely nailed down with wrought-iron nails 4 ins. long. After several miles of this description of road had been made, long granite slabs were substituted for the cedar cross-pieces and the yellow pine stringers."

"The iron used for rails was $\frac{1}{2}$ to $\frac{5}{8}$ in. thick by $2\frac{1}{4}$ to $4\frac{1}{2}$ ins. wide. The heads of the nails or spikes holding it down were countersunk in it."

One would judge, from the varying thicknesses and widths, that the specification and inspection were not very rigid.

Notwithstanding these strap-rails being "securely nailed down," it was found that traffic would loosen them, with the final result of their turning up as the wheels passed over them, and forming what were called "snake heads." These would occasionally tear through the bottoms of the cars, and cause more or less inconvenience if not danger to the passengers. So the American engineers again turned to England, where the same difficulties had led to the invention of rails of different sections. It is believed that the first one was the fish-bellied rail, invented by John Birkinshaw, of the Beddington Iron Works, and patented in October, 1820. This rail was held in cast-iron chairs by side keys or wedges. The Baltimore and Ohio Company soon afterward imported some of these rails.

The Stockton and Darlington, and its follower, the Liverpool and Manchester, which was opened in September, 1830, were principally laid with rails of the Birkinshaw type. The Stockton and Darlington also had a few cast-iron fish-bellied rails.

The Clarence rail was another English invention and was considered an improvement on the Birkinshaw. Rails of that pattern were imported into America for the Allegheny Portage Railroad, built by the State of Pennsylvania over the Allegheny Mountains to connect the canals on either side of them. This road was opened in 1833.

In 1834 the Columbia and Philadelphia Railroad was opened. Part of this road was laid with flat rails, but on the greater part the Clarence rails were used. On both roads the rails rested on stone blocks. These roads were in after years absorbed by the Pennsylvania Railroad.

Another English section was the H-rail, which rested in a chair. These were imported, and used on some of the roads. Still later, came the U-rail, known in Wales as the Evans patent, and believed to have been first rolled at the Dowlais Works.

Some of the flat strap-rails were made in America, but all the sectioned ones were imported. Some attempts were made to use American cast-iron rails, but with unsatisfactory results. It was not until

Mr. Hunt. 1844 that the manufacture of sectioned wrought-iron rails was begun in America. A rolling mill was built in 1843 by the Mount Savage Rolling Mill Co., at Mount Savage, Allegheny County, Md., expressly to make rails. Operations commenced in 1844, and for their first rail, which was of the U-section, they were awarded a silver medal by the Franklin Institute of Philadelphia. The rail weighed 42 lbs. per yard. About 500 tons were laid in 1844 on the road then being built between Mount Savage and Cumberland, Md. A short time later they rolled some 52-lb. rails for a road between Fall River and Boston, and in 1845 and 1846 they rolled T-rails. This mill, after being long abandoned, was finally dismantled in 1875.

The T-rail was generally known in Europe as the "Vignoles" rail, after Charles V. Vignoles, an English railroad engineer, who introduced its use there. But it was really invented by Robert L. Stevens, of Hoboken, N. J., President and Engineer of the Camden and Amboy Railroad.

In 1845, the Montour rolling mill, at Danville, Pa., was built expressly to roll T-rails, and in October of that year there was rolled in that mill the first rail of that section made in America.

In 1846, T-rails were rolled by the Boston Iron Works, Boston, Mass.; by Cooper and Hewitt's mill at Trenton, N. J.; by the New England Iron Company, Providence, R. I.; by the Phœnix Rolling Mill Company, Phœnixville, Pa.; by the Great Western Iron Company, Bradys Bend, Pa.; and by the Lackawanna Iron Works, Scranton, Pa.

In the following years the manufacture was taken up by other companies; but owing to the commercial conditions caused by the severity of foreign competition, early in 1850, only two out of the fifteen rail mills in the United States remained in operation.

These early rails were all short; none over 15 ft. long. As the difficulties of manufacture were overcome and the science of track laying progressed, the length was gradually increased until 21 ft. was reached, and was considered the limit. It was not until about 1859 that railway engineers would accept those of greater length.

The first 30-ft. rails rolled in America were made by the Cambria Iron Company, Johnstown, Pa., in 1855; but they could not find sale for them and they were finally used by that company in their mill yards. The first 30-ft. rails to fill an order were rolled by the Montour Company, in January, 1859, for the Sunbury and Erie Railroad Company.

The rolling of iron rails was attended with many difficulties. If the pile of bars was not heated to a sufficiently high degree, the welds would not be perfect; and if heated too highly, the iron would crack in the process of rolling and yield an imperfect product. If the metal was too soft, although the rail might be free from flaws and bad welds,

it would wear out rapidly under traffic. Under all circumstances it was important that the rolling process should be completed as quickly as possible so that the reductions should be made while the iron had lost little of its heat. This, together with some local conditions, led to the invention by John Fritz, Hon. M. Am. Soc. C. E., of the three-high rail train. Three-high sets of rolls had been used for many years in making merchant bars, but it required the application of the "Fritz Yielding Hanging Guides and Driven Feed Rollers," to make them practical for rail rolling. This improvement was put into successful operation at the Cambria Mills in 1857. It has ever since remained as the typical American rail mill. Since the introduction of steel rails there have been several two-high reversing mills on the English plan, used in America; in fact, two of this kind are now running. But the three-high is the American mill, and has permitted the tremendous production which has been attained in later years. Mr. Hunt.

The early mills required the work of handling the material as it passed through the rolls to be done by manual labor, through the use of tongs and hooks. Probably the rolling of iron piles with their necessarily peculiar handling, would have indefinitely continued this, but with the use of solid steel blooms, the troubles lessened and made possible the introduction of automatic machinery. The tong and hook system necessitated the employment of 15 to 17 men, and the production of steel rails was limited to not over 250 tons per turn. Automatic machinery revolutionized this, both as to number of men employed and the possibilities of production.

It was the writer's fortune to introduce the first driven rail-mill tables, those in the works of the Albany and Rensselaer Iron and Steel Company, Troy, N. Y., in March, 1884. These were in front of the finishing rolls, and worked so well that an automatic arrangement was soon after placed in front of the roughing rolls. This latter arrangement was more particularly designed by Mr. Max M. Suppes, then the master mechanic of the works, and now the general manager of the Lorain Steel Company, Lorain, Ohio. Naturally, these devices were protected by letters patent. From this start other inventions were made, and many improvements by other American engineers have followed, until the present American rail mill, capable of turning out 50 000 tons of finished rails per month, has been developed.

It was the writer's fortune to become connected with rail making in 1856, and among his earliest recollections is the statement that the users of rails had in service certain makes which had been and were giving results impossible to be obtained from any of more recent manufacture. How familiar that statement must sound to many of you, and as of recent date!

Then, as now, the question demanded an answer, and many sought for the solution.

Mr. Hunt. The first iron rails were made from straight puddled bars. These bars were about 1 in. thick and were placed one upon another, until a pile of sufficient weight and height was formed; the pile was then reheated and rolled into rails. And it was to the formation of that pile that inventive genius was applied.

From an investigation of the fracture of some of the rails which had given satisfaction, it was discovered that the pile of bars from which they had been rolled, had been entered in the rolls edgewise, thus bringing the line of welds between the bars in a vertical instead of horizontal position. This presented a different structure to the wheel wear, and seemed to be logical. Based on that supposition many rails were so rolled, and the writer believes that the scheme was patented.

Where the rail was rolled with the layers of the pile in a horizontal position, particular attention was given to the character of the top bar, which would, of course, form the wearing surface of the rail. Cold-short or granular iron was used for it, while the remainder, or at least the flange of the rail, was of fibrous iron.

At one time a rail with a puddled-steel head—or rather with the top bar of the pile of puddled steel—found much favor, but, owing to the difficulty of obtaining uniformly good welds, the results were not satisfactory. Some of these so-called steel-headed rails had the top bar of what was known as silicon steel.

Another plan, on which much money was spent, was to hammer a puddled ball, or weld two puddled balls together, under a steam hammer, and draw them into a slab 2 to 2½ ins. thick, which was used on the top of the rail pile. Under an order from the Pennsylvania Railroad Company, the Cambria Iron Company, in whose employ the writer was then serving, erected a special steam hammer, and made several thousand tons of such rails. Their service was somewhat disappointing, and the practice was abandoned.

At that time, as since, commercial conditions controlled. The railroads had the worn-out rails on their hands, and, regardless of whether or not the practice would give satisfactory results, they adopted a system of having the old rails re-rolled into new ones. At first a certain percentage of new iron was specified, but as the necessities for immediate economies increased, that demand was eliminated from the contracts, and the new rails were composed entirely of the old ones. The best practice was to make a pile of old rails, break it down into bars, which were piled upon each other, and then rolled into rails. But presently this was found to be too expensive to successfully meet the cry for cheaper rails, and only the top and bottom of the piles were formed from re-worked iron, the center being composed of from three to six pieces of old rails.

From the many re-workings, the cheapening of the process of manufacture, and the increasing demands of traffic, the wear of the

iron rails become more and more unsatisfactory, until it seemed as Mr. Hunt.
though, from that cause alone, the limit of railway development had been reached. Such situations frequently occur in earthly affairs; and seldom if ever has the occasion failed to be met by a solution of its difficulties. In this case, came the invention of Bessemer.

It is a historical fact that the first rail ever made from Bessemer steel was placed on the Midland Railroad, of England, early in 1857, at a point where iron rails had sometimes to be renewed within three months; and it remained there until June, 1873, some sixteen years, during which time about 1 250 000 trains and any number of detached engines and tenders passed over it.

We all realize that without such an innovation as Bessemer's, the subsequent tremendous expansion of railway development would have been physically impossible.

Railway managers were timid about using steel rails, and in America many attemps were made to produce a satisfactory rail having an iron base and web, with a steel-capped top. None was satisfactory, and the Bessemer steel rail soon conquered the situation.

The first steel rails laid down by an American railroad were imported by J. Edgar Thomson, President of the Pennsylvania System.

The first to be manufactured in America were rolled at the mills of the North Chicago Rolling Mill Company, Chicago, Ill., on May 24th, 1865, from ingots produced in experimental steel works at Wyandotte, Mich. They were not many in number, and were made on the regular iron rail rolls of the mill. Several of the rails were put in local railway tracks and gave good service.

The first production of steel rails in the United States, on a commercial order, was at the Cambria Iron Company's mill, in August, 1867, from ingots made by the Pennsylvania Steel Company, near Harrisburg, Pa. The converting works of that company were completed some time in advance of its rail mill, which led to an arrangement under which the ingots were sent to Johnstown to be hammered into blooms which were then reheated and rolled into rails. The steel was made under the management of the late Alexander L. Holley, M. Am. Soc. C. E., then in charge of the Pennsylvania Steel Company. George Fritz was the Chief Engineer and General Superintendent of the Cambria Iron Company, and Alexander Hamilton, Superintendent of the rail and other mills, while the writer was in direct charge of the steel department.

It is a matter of some interest, that the ingots were drawn down by the steam hammer which had been installed some years before to make the hammered iron slabs for the Pennsylvania Railroad Company's rails. From this time, the production in America of Bessemer steel rails increased rapidly.

Mr. Hunt. For a time after the starting of the Pennsylvania Steel Company's Bessemer Works, the ingots were cast from the top, on the then-accepted English plan. Mr. Holley's mind was not so constituted that he could long follow any beaten track without an effort to do better work on some other line. Thus, he introduced the bottom-casting of ingots; pouring the steel into a central octagonal mould about 14 ins. diameter at the bottom and 10 ins. at the top, from the bottom of which the metal flowed through connecting gates into four surrounding moulds 8½ ins. square. This plan was adopted after consultations with Mr. George Fritz, who had rolls turned to take the 8½-in. ingots. The central or sprue ingots were hammered into blooms. It was found that the small ingots rolled satisfactorily, while, on the contrary, the central ones cracked badly during working.

This led to much discussion and consultation among the operative officers of the Cambria Company and Mr. Holley, the result of which was that John E. Fry, then superintendent of the Cambria Iron Company's iron foundry, suggested the use of a rammed-up center sprue, 4 ins. in diameter, connecting through fire-brick gates with surrounding ingots; the sprue and gates to be treated as scrap. This plan answered admirably.

While in charge of the experimental Bessemer Works at Wyandotte, Mich., in the interest of the Cambria Iron Company, the writer had developed a manner of bottom-casting ingots. Mr. Holley, having protected his plan by a patent, Mr. Fry and the writer united in patenting theirs, and their interests and those of Holley were consolidated. For some years after this, practically all bottom-casting of ingots in America was licensed under these patents. After a time the price of rails became so much reduced that the loss incident to the scrap of the center sprue and bottom gates made in bottom-casting became a serious matter, and while it was and is impossible to cast as sound, and hence as good, ingots from the top, the better plan was abandoned.

At first, all the American Bessemer's works pursued the English plan of reducing the ingots to blooms under steam hammers. This was so at the Pennsylvania and Troy Works. The success in rolling the 8½-in. ingots at Johnstown led to the invention of the American blooming mill, and this soon completely superseded the steam hammer in rail making.

This idea originated with Mr. George Fritz. Holley and he were intimate friends and exchanged views freely. Holley had severed his connection with the Pennsylvania Steel Company, and returned to the Troy, N. Y., Works. There he built a three-high blooming mill. While it had tables, their rollers were not power-driven, and the ingots had to be pushed into the rolls and turned over on the tables by hand. Soon after, George Fritz built a blooming mill at Johns-

town, in connection with a Bessemer converting plant, and put into use his patented ideas of driven rollers, hydraulically controlled movable rolls, and a "turning over and sliding from pass to pass" device, christened by the mill hands a "go-devil," which permitted the economical handling of larger ingots. This was the birth of the American blooming mill. In perfecting his plans George Fritz had the benefit of the advice of his brother John, then manager of the Bethlehem, Pa., Works. Mr. Hunt.

Perhaps these details apply more to rolling-mill practice than to the rails themselves; but the writer thinks that they have played a most important part in relation to the character of steel rails, and are pertinent to the subject. Holley started the innovation by which the production of steel ingots has been increased so greatly. Fritz gave the blooming mill, which would not only take care of all that was sent to it from the converting works, but like Oliver Twist, ask for more; and the late Capt. William R. Jones, Robert Forsyth, M. Am. Soc. C. E., and several others, built rail mills which were not satisfied with the amount of steel sent to them by any blooming mill. This has all been magnificent. It has made possible undreamed-of low prices for steel rails. It has helped to build railroads, but has it improved the quality of the rails produced?

Steel rails, when first manufactured, replaced iron rails, which, through their deteriorated quality and the increased duty demanded of them, were giving most unsatisfactory service. Some of the early steel rails failed, but most of them were so much better than the best of their predecessors that such failures did not excite adverse comment. They were of what would now be considered light sections, and thus in their production from the 6 × 6-in. or 7 × 7-in. blooms from which they had been rolled, had received much work, and at a comparatively low temperature. In the writer's judgment the greatest factors in the production of good rails are covered by the words "work and temperature." All steel men know that work at high heats does not change the grain of steel at all in proportion to work given at lower temperatures.

For years after the introduction of steel rails a 65-lb. per yard section was considered a heavy one. In fact, in America it was the heaviest used, and much the largest percentage was not over 60 lbs. These were rolled from 7 × 7-in. blooms. The ingots from which the blooms were made were generally 12 × 12 ins. After the bloom was formed it was examined carefully after becoming cold, and all cracks and mechanical imperfections were chipped out. Then, after slow heating, with care to avoid too high a temperature, the blooms were rolled into rails by light reductions. While this was being done, if a defect showed itself, the process was stopped until it was chipped out. Now, this slow work at a moderate and steadily decreasing tempera-

Mr. Hunt. ture, resulted in a fine grained metal, which, of necessity, no matter what may have been its chemical composition, would give greater resistance to the wear of traffic than could be possible from the coarser grained steel which is in the head of the heavier and more rapidly rolled sections of to-day.

By waiting long enough the things of the past always become the best. That is, provided the past is not examined too closely. It must be remembered that the early rails replaced a much inferior article; in fact, created a revolution in railway maintenance of way. Hence, if a few from any cause failed, it excited little comment; they were quietly replaced by others. After a while these failures were forgotten, and the whole lot of existing rails was instanced as an example of what rails should be. Another thing which must not be overlooked is, that the early steel rails had the ultimate stress of traffic applied by slow degrees. In other words, the traffic to which they were subjected when first put in service was for them light duty. Heavier rolling stock, faster and more frequent trains, came gradually. The old-time rails, which are in these later days so reverently mentioned, had been subjected to a cold-rolling process before being given their severest task. To-day an 80-lb. rail is hardly cold before a 175 000-lb. locomotive, hauling 100 000-lb. capacity cars at 35 miles per hour; and limited expresses of heavy Pullmans at 60 miles per hour are thundering over it.

The details of manufacture of steel rails changed, not only in America, but also in England and other countries. This had to be, and it would to-day be as impossible to return to the earlier methods as to restore the service of stage coaches.

In 1876, the writer presented a paper at the Philadelphia meeting of the American Institute of Mining Engineers, on "The History of the Bessemer Process in America," and with great pride chronicled the fact that the North Chicago Bessemer Works, under the management of Robert Forsyth, had, in a single month, produced 6 457 gross tons of ingots, and that it led the world's records. These ingots were all rolled into rails. To-day, the North Chicago Converting Works and rail mill are abandoned, their places having been taken by the present South Chicago plant of the Illinois Steel Company, in which rail mill the largest month's production has been 58 103 gross tons of standard-sectioned rails.

The Edgar Thomson Works of the Carnegie Steel Company has made 47 074 gross tons of rails in a calendar month. Other American mills are turning out a large tonnage, but it is believed that the foregoing at present hold the record.

While the faster work of modern practice has somewhat altered the character of the steel in rails, it must not be assumed that the product has been increased without any regard to other considerations.

That is not true; on the contrary, the outward character or finish of the rails has been improved to a radical extent. While working fast, the improved machinery is also reliable, and the care exercised in keeping true to section, square sawing, accurate drilling and straightening of both line and surface, yield results which it would have been impossible to obtain in the earlier days. In fact, the requirements of the railways, in consequence of increased weight and speed of traffic, etc., have made it imperative that such finished rails should be given them. Mr. Hunt.

It is not desired to draw any invidious comparisons, but, in the writer's judgment, American makers are to-day not only turning out the most rails, but at the same time the best finished ones now produced. Moreover, foreign rails, imported into the United States and Canada during late years, have not worn any better than American rails.

It has been stated that examining into the past sometimes disproves assumptions. So that, while in the earlier days rail steel and rails were made with all the time and care which has been described, all the rails produced were not satisfactory. In fact, the experience of the Pennsylvania Railroad was such that their chemist, Charles B. Dudley, M. Am. Soc. C. E., made an elaborate investigation into the chemical composition of their satisfactory and unsatisfactory rails. His deductions were presented to the American Institute of Mining Engineers in October, 1878, and elicited a memorable discussion. Dr. Dudley concluded that rail steel could be too hard for good service, even though the rails did not actually break. His conclusions were in favor of chemically softer metal. Some of those discussing the matter thought and stated that the size of the rail sections should not be ignored.

As the accredited authority of the great Pennsylvania Railroad, Dr. Dudley's views carried added weight, and it resulted in a demand in America for softer rails. The writer thinks that no one to-day will attempt a defence of that position. In fact, the practice did not long prevail. But it cannot be safely claimed that the present rails, whether made in America or imported from Europe, are giving absolutely satisfactory results. They are permitting the accomplishment of work which, but a little while ago, would have been considered an impossibility. Still, if engineers had ever been absolutely satisfied with that which was, progress would have halted. Heavy-sectioned rails which will yield better results than those now being obtained are needed. Our railway organizations have generally become so situated financially, that they need no longer be limited to the immediate present in the policies of their administrations.

In the old countries, railroads were built because there was a population whose needs demanded them. In this country, they were often built because there was a tremendous amount of country and no popu-

Mr. Hunt. lation. This led to cheap construction, but while we still have plenty of room for more people, our country has become rich enough to justify the best of railroads, and, in fact, imperatively demands them. The successful operation of the roads themselves can be only on such a basis. As the railroads increased in number, and their requirements varied in accordance with their traffic and profiles, and being constructed and operated by many different men, it was natural that not only the weight of rails used, but their sections, should differ. In fact, almost every road had its own particular section. The variations between many were slight, but sufficient to necessitate the use of special rolls in their manufacture. This had become so pronounced, and caused so great an investment of capital on the part of rail makers and loss of time in changing from one section to another, etc., that Mr. Holley, in his characteristic progressive spirit, attacked the situation in a paper presented to the American Institute of Mining Engineers, in February, 1881. In this he stated that, answering his inquiries, the eleven Bessemer mills then making rails in the United States had sent him drawings of 188 patterns which were considered standard ones, and that 119 patterns of 27 different weights per yard were regularly manufactured. Mr. Holley gave drawings of many of these and pointed out the absurdity of some of the variations. His paper attracted wide interest, but it was a difficult matter to reach. However, its discussion was continued by others later. Mr. P. H. Dudley, who had devoted much time to the study of the wear of rails, and who invented a recording car for the examination of the rails, contributed papers on the subject to both the scientific press and societies. J. D. Hawks, M. Am. Soc. C. E., then Chief Engineer of the Michigan Central Railroad, advocated certain sections, as also did D. J. Whittemore, Past-President, Am. Soc. C. E., and the writer also presented a series of sections, in a paper read before the American Institute of Mining Engineers, February, 1899.

This Society appointed the committee on "The Proper Relation to Each Other of the Sections of Railway Wheels and Rails," which performed its duties in a thorough manner; and following and resulting from its reports, the Society appointed a committee to consider and recommend a series of Standard Rail Sections. This was in 1891, but the final report of the Committee was not made until June, 1893.* During the intervening years the members of the committee had worked faithfully; consulted personally and by correspondence with many of the chief engineers of the railroad systems of the country, and agreed upon all points but one. One member, Mr. George S. Morison, differed with the Chairman, Mr. G. Bouscaren, and with Messrs. Foster Crowell, Virgil G. Bogue, S. M. Felton, J. D. Hawks, E. T. D. Myers, Samuel Rea, Thomas Rodd, A. M. Wellington, F. M.

* *Transactions*, Am. Soc. C. E., Vol. xxviii.

Wilder, and Robert W. Hunt, as to the width of heads of the proposed sections, and, on that point, made a minority report. Mr. Hunt.

As secretary of the committee during the latter part of its work, the writer knows the difficulties and labors of the task, and naturally, is gratified to know that the recommended rail sections are to-day practically the standard ones for American railroads. During 1899, quite 75% of all the rails rolled by American rail mills were of what are commercially known as the American Society sections.

Geographical and commercial conditions must govern. When the Bessemer process was first introduced in America, imported English pig irons were used in making the steel. American irons were experimented with and gradually displaced foreign irons. This practice first prevailed in the works located west of the Allegheny Mountains. They soon relied entirely upon charcoal pig, made from Lake Superior and Missouri ores. This was much higher in phosphorus than the English irons, but the results obtained from it were so satisfactory that the investigation continued and extended to the use of American mineral fuel irons, both anthracite and coke. After a time these completely displaced both foreign coke and American charcoal brands, in both western and eastern works.

It happened that, while the most available western ores contained percentages of phosphorus fully up to the limit possible for Bessemer uses, the cheapest eastern ores were quite low in that obnoxious element. Hence, it has been and is a fact, that some of the rail makers located east of the Alleghenies can produce rails low in phosphorus contents, while using the lowest priced pig metal. The opposite is true of the western mills. These geographical and commercial conditions have led to the use of entirely distinct chemical specifications in the two districts—at least by some of the leading makers in those districts.

The heavier equipments and higher speeds required more rigid road beds, which could only be obtained by heavier sectioned rails. These were gradually adopted. It was naturally expected that as the sections were increased so would the resulting amount of service yielded by the rails. From the very first, the results obtained were disappointing, and the writer doubts whether we will ever succeed in getting results as satisfactory as those yielded by the lighter sections. As the area of the section is increased, so, of necessity, will the work upon the steel in forming it be decreased; and as the resulting mass is enlarged, so will the amount of heat retained in it at the time of the final reduction through the rolls be increased. In the writer's judgment, it will be found that the most satisfactory results will be obtained by so modifying the rolling system that the final pass (or better, passes) shall be given after the temperature of the partially formed rail has been lowered. This is not by any means a new idea, but as yet it has not been carried out in a manner calculated to obtain the best results.

Mr. Hunt. Some years ago, Mr. F. A. Delano, now Superintendent of Motive Power of the Chicago, Burlington and Quincy Railroad, in the interest of that company, had some rails rolled at the South Works of the present Illinois Steel Company, then owned by the North Chicago Rolling Mill Company, on such lines and under his personal supervision. Unfortunately, these rails were of a peculiar section, which was not continued, but the writer believes that the wear of the metal itself was encouraging.

The satisfactory wear being given by rails renewed by the "McKenna Process" at the Joliet and Kansas City Mills of the McKenna Steel Working Company, bears very strongly on this point. Mr. McKenna takes rails which have become unfit for further service in main line tracks, from having become rough in surface, through flow of metal, or other causes; or which have become curve-worn on the side of the head; and, after carefully removing any fins which have been formed on the upper edges of the heads by metal flow, charges them into a long furnace, and, when heated to not more than 1 500° Fahr., they are drawn from the furnace by a mechanical contrivance which at the same time removes any scale which may have formed on their surface, and slightly upsets or flattens the section. The rail is then passed through a set of forming rolls, from which it is carried forward to another set, in which it is given a finishing pass. The rail is then sawed hot, and cold-straightened and drilled in the usual manner. And while the section has been somewhat reduced, the original finishing sections and heights have been maintained.

Now, the steel has been given finishing work at low temperature, and examination has proven that the grain of the metal in the head of the rails has been "fined." But, more important than all, the wear of the renewed rails is promising to be much more satisfactory than that obtained from new rails of heavier sections. This treatment of rails is no longer in an experimental state, as it is over five years old, and there are nearly 100 000 tons of renewed rails in service on the Chicago, Milwaukee and St. Paul; Atchison, Topeka and Santa Fé; Wabash and other large systems. One chief engineer, on whose road there are many of these rails, says: "No rail ought to be used at all until after it has been renewed."

The writer has already stated that, owing to geographical, and hence commercial, conditions the chemical specifications under which rails are made differ east and west of the Allegheny Mountains. In the Scranton and Bethlehem District what are known as the New York Central and Hudson River Railroad specifications, which were originally formulated for that road by Mr. P. H. Dudley, are regarded as the standard; while for the mills west of the Alleghenies, a different formula is followed. Some of the main railway systems insist on buying under their own chemical specifications, no matter where made.

Mr. Hunt.

The writer has gone on record so often, as believing that in the absence of work at low heats, incident to the present method of making heavy-sectioned rails, it is important to increase the carbon with the section to as great an extent as the phosphorus present will permit, without incurring risk from breakage, that it seems unnecessary to repeat the arguments.

At the Atlanta meeting of the American Institute of Mining Engineers, in October, 1895, the writer presented a set of specifications for "Steel rails of Heavy Sections Manufactured West of the Alleghenies." In accordance with these specifications thousands of tons have been made and used with satisfactory results. During the last two years the western makers have declined to limit the phosphorus to less than 0.10 %, but, in fact, have been making steel with a fraction less than that amount—say 0.09 to 0.096 per cent. And he regrets to say that in many cases they insist that the amount of carbon shall be less than that which he has advocated. He believes, however, that gradually, higher carbon will prevail, and certainly has not had any cause to change his mind on the subject. His experience as a steel-rail maker, and as an observer of the wear of steel rails of many sections and diverse chemical composition, leads him to advocate: *First*, work, after careful heating of the steel, and continued until its temperature has been much reduced. *Second*, that the carbon percentages shall be increased in proportion to the increase of rail section, the ultimate amount being, of necessity, limited by the contained percentage of phosphorus. In all cases he advocates the use of drop tests, on samples from each heat of steel.

At present many of the American railway engineers use the drop test, but none of them demands the static or tensile tests insisted upon by so many engineers of other countries; nor does the writer think there is any necessity for these latter. The chemical analyses and drop tests are all sufficient.

As a matter of record, the writer gives the chemical formulas contained in his specifications of 1895, in accordance with which, as stated, thousands of tons of rails have been made and have given good results. And while at present the western makers decline to limit their steel to 0.085% phosphorus, the writer certainly sees no reason to decrease the carbon. In other words, so many rails have been made and proven safe with quite as much carbon as given in these specifications, and with 0.10% phosphorus, that the writer does not think the former element should be made less, certainly not until the details of manufacture have been changed.

The standard specifications of the Louisville and Nashville Railroad Company are also given, as they cover both Bessemer and basic open-hearth steel rails.

The so-called New York Central and Hudson River Railroad Com-

Mr. Hunt. pany's specifications are also appended; and the present standard specifications of the western rail mills.

Robert W. Hunt's Specifications.

Sec. 8.—The carbon in the 70-lb. section shall not be below 0.43% nor over 0.51 per cent. In the 75-lb. section, not less than 0.45% nor over 0.53 per cent. In the 80-lb. section, not less than 0.48%, nor over 0.56 per cent. In the 90-lb. section, not less than 0.55%, nor over 0.63 per cent. In the 100-lb. section, not less than 0.62%, nor over 0.70 per cent.

The phosphorus shall not exceed 0.085 per cent.

The silicon shall not be below 0.10 per cent.

The remainder of the chemical composition of the steel to be left to the maker's judgment.

Louisville & Nashville Railroad Specifications.

The steel used for rails shall contain carbon as follows: For 58¼-lb. steel rail from 0.42 to 0.52 of 1%; for 70-lb. steel rail 0.47 to 0.57 of 1%, and for 80-lb. steel rail 0.55 to 0.65 of 1%, and not more than 0.085 of 1% of phosphorus, or 0.07 of 1% of sulphur. Silicon, 0.15 to 0.20 of 1 per cent.

When the steel used for the rails has been made by the basic open-hearth process, it should be of the following chemical composition: For 58¼-lb. steel rail, from 0.45 to 0.52% of carbon; for 70-lb. rail, 0.50 to 0.57%; for 75-lb. rail, 0.55 to 0.60%; for 80-lb. rail, 0.62 to 0.67 per cent. The steel used for all rails shall contain silicon from 0.10 to 0.20%, 0.15% being preferred; manganese, 0.90 to 1%; phosphorus, not to exceed 0.05%; sulphur, not to exceed 0.05 per cent.

New York Central & Hudson River Railroad Specifications.

	65-lb.	70-lb.	75-lb.	80-lb.	100-lb.
Carbon	0.45 to 0.55	0.47 to 0.57	0.50 to 0.60	0.55 to 0.60	0.65 to 0.70
Silicon	0.15 to 0.20	0.15 to 0.20	0.15 to 0.20	0.15 to 0.20	0.15 to 0.20
Manganese	1.05 to 1.25	1.05 to 1.25	1.10 to 1.30	1.10 to 1.30	1.20 to 1.40
Sulphur	0.069	0.069	0.069	0.069	0.069
Phosphorus	0.06	0.06	0.06	0.06	0.06
Rails having carbon below will be rejected	0.43	0.45	0.48	0.53	0.60
Rails having carbon above will be rejected	0.57	0.59	0.62	0.65	0.70

Specifications of the Western Rail Mills.

Section 1.	50-lb. up to 60-lb.	60-lb. up to 70-lb.	70-lb. up to 80-lb.	80-lb. up to 90-lb.	90-lb. up to 100-lb.
Carbon	0.35 to 0.45	0.38 to 0.48	0.40 to 0.50	0.43 to 0.53	0.45 to 0.55
Phosphorus	not over 0.10	not over 0.10	not over 0.10	not over 0.10	not over 0.10
Silicon	not over 0.20	not over 0.20	not over 0.20	not over 0.20	not over 0.20
Manganese	0.70 to 1.00	0.70 to 1.00	0.75 to 1.05	0.80 to 1.10	0.80 to 1.10

ALBERT LADD COLBY, M. Am. Soc. Mech. Eng. (by letter).—Mr. Hunt very appropriately opens the discussion by remarking that: Mr. Colby.

"As America came to Great Britain for the rails used on her first railroads, it is perhaps fitting that this discussion should be presented to the Society in London, and at its first English meeting."

To this the writer would like to add that as America now exports rails to British Colonies, it is also fitting, perhaps, that at this first English meeting, a few words be addressed to English engineers in friendly criticism of foreign rail specifications, and a few practical suggestions given as to modifications which can safely be made in their specifications without any danger of obtaining an inferior product from American rail mills, and with the decided practical advantages of lower prices and more prompt delivery.

No discourtesy is intended to our hosts, the Institution of Civil Engineers, in this evident effort to advertize American steel. The halls of our technical societies, and the columns of our technical press are just as freely open to our English friends, and no one is quicker to appreciate and adopt foreign ideas, patents or products than the American engineer, and the American purchasing agent.

The following official statistics show the rapid increase in the tonnage and value of steel rails exported from America in recent years:

	Gross tons.	Value.
1894	12 229	$323 880
1895	8 807	222 661
1896	72 503	1 712 716
1897	142 808	2 949 901
1898	291 038	5 787 384
1899	171 272	6 122 382

In the following brief review, the requirements of foreign rail specifications will be compared with those contained in a proposed standard American specification for rails, recently framed by Committee No. 1 of the American Section of the International Association for Testing Materials, and which specification will be presented by this Committee at the Annual Meeting of the American Section, in October, 1900, for discussion and adoption as the standard American specification.

1. THE PROCESSES OF MANUFACTURE SPECIFIED.

In a review of forty-one foreign rail specifications, eight were found which limit the manufacturer to the use of "The best English or Spanish hematite pig iron and charcoal spiegeleisen." This requirement, of course, cannot be complied with in American mills, nor in the present state of the art, does it seem a necessary stipulation in any case, particularly as, if strictly interpreted, it seriously limits competition.

Mr. Colby.

2. The Chemical Properties Specified.

(*a*) *Carbon.*—In 63% of the forty-one foreign specifications reviewed, limits in carbon are specified, and in the majority of these cases a range of carbon of from 0.10 to 0.15% is given. This is in general accord with American practice, except that the proposed American standard specification increases the carbon gradually with the increased weight of section, specifying in each case a range of 0.10 per cent. This is in accord with Mr. Hunt's suggestion that "The carbon percentages shall be increased in proportion to the increase of rail section."

The actual percentages of carbon given in foreign specifications will not be discussed in detail. The proper structure to insure safety and wearing qualities is what is desired in rails; and as the percentages of carbon, and other hardening elements which will give this structure, vary with the finishing temperatures, the chemistry must be changed to suit the varying conditions of manufacture. Some American rail mills have recently been successful in experiments looking toward the finishing of rails at a lower temperature than at present in vogue in America, without diminishing the present output. This recently patented improvement is in accord with Mr. Hunt's suggestion that:

"In the writer's judgment, it will be found that the most satisfactory results will be obtained by so modifying the rolling system that the final pass (or better, passes) shall be given after the temperature of the partially formed rail has been lowered."

In short, an international standard composition for rails is utterly impracticable, and it is useless for any country, or any engineer, to assert that their composition is the best, and, therefore, should be adopted in all countries.*

(*b*) *Phosphorus and Sulphur.*—Thirty-two per cent. of the foreign specifications reviewed, called for a phosphorus and sulphur of 0.06% or below. Rails meeting these requirements will be furnished in America, at, of course, a considerably higher price than charged for rails of 0.10% phosphorus, which latter forms by far the largest proportion of the product of American rail mills. Many thousands of tons of American rails of 0.10% phosphorus have given satisfactory service under severe conditions. The increased cost of 0.06 or 0.08% phosphorus rails, over that of rails containing 0.10% phosphorus does not, in the opinion of many competent judges in America, obtain a correspondingly safer or better rail from American mills.

Mr. Hunt states that:

"Owing to geographical, and hence commercial, conditions the chemical specifications under which rails are made differ east and west of the Allegheny Mountains. In the Scranton and Bethlehem District what are known as the New York Central and Hudson River Railroad specifications, which were originally formulated for that road by Mr.

* See C. P. Sandberg's paper: "The danger of using too hard steel rails." *Journal Iron and Steel Institute*, No. 2, 1898, pp. 76-110; especially p. 106.

P. H. Dudley, are regarded as the standard; while for the mills west of the Alleghenies, a different formula is followed." Mr. Colby.

The Bethlehem District referred to is no longer a producer of rails, and the writer happens to be in a position to know that the New York Central and Hudson River Railroad Company has very recently abandoned the 0.06% phosphorus limit recommended by Mr. P. H. Dudley, and has purchased rails of 0.10% phosphorus. It can hence be safely asserted, that, as far as tonnage is concerned, practically all American roads now use rails of 0.10% phosphorus.

(*c*) *Silicon and Manganese.* —About one-half of the foreign rail specifications examined mentioned limits in silicon and manganese. The silicon specified varies from 0.06% to 0.25 per cent. The manganese varies between the wide limits of 0.40% and 1.20 per cent. American mills would not be inclined to furnish rails on a specification limiting the manganese to 0.40%, the requirement mentioned in one specification examined. The American standard specification for rails states that silicon shall not exceed 0.20%; and gives a range of 0.30% in manganese, the limits varying with the increased section between 0.70 and 1.10 per cent.

In general, it may be stated that the strict adherence, by a foreign engineer, to a specified complete chemical composition, independent of the varying conditions of manufacture, and the rejection of large lots of rails solely because the sample, assumed to represent the lot, exceeds in some respects the specified chemistry, compels his client to pay much more than is necessary to obtain a first-class rail to-day from American mills.

3. Samples for Analysis.

Some foreign specifications require that every heat of rail steel shall be analyzed for phosphorus and arsenic, and stipulate that if the phosphorus and arsenic exceed 0.06% the heat is not to be used. It has been proved conclusively that arsenic should not be classed with phosphorus in its effect upon the physical qualities of steel.* Furthermore, as there is no rapid method by which both phosphorus and all of the arsenic can be determined, the ingots of each blow could not be held for analysis, but must be rolled into rails and subsequently rejected if the analysis shows a phosphorus and arsenic of over 0.6 per cent. This burden, with the unnecessarily frequent analysis demanded, will not be assumed by the American manufacturer without substantial remuneration, and will more often result in a refusal to bid unless this requirement of the specification be withdrawn.

Another very arbitrary clause in certain foreign specifications, states that each 500 tons of finished rails will be accepted or rejected on the

* F. W. Harbord and A. E. Tucker, *Journal, Iron and Steel Institute*, No. 1, 1888, pp. 183-196, and No. 1, 1895, pp. 131-132 and 127-128.
J. E. Stead, *Journal, Iron and Steel Institute*, No. 1, 1895, pp. 77-140.

Mr. Colby. analysis of a piece of one rail, selected by the engineer; said analysis to be made, at the contractor's expense, by an independent chemist, selected by the engineer. In American mills the rails pass over several cooling beds, and are drilled on many sets of presses. Without great additional labor, therefore, it is impossible to land the finished rails of consecutive heat numbers in 500-ton piles in the yard, to await the complete analysis of the independent and probably distant chemist. In the light of all that has been published, and candidly admitted by the manufacturer, as to the unavoidable variation in composition even of the different parts of a section of rail, to say nothing of the variation of different rails of a blow, how can an engineer conscientiously accept or reject 500 tons of rails from the analysis of a single rail ?

4. The Physical Properties Specified.

(*d*) *Tensile Strength and Elongation.*—With a specified chemical composition, a drop test, and, in most cases, a dead-weight test also, tensile strength and elongation is an unnecessary requirement. In fact, when carbon, and other hardening elements are specified, and strictly adhered to, the tensile strength and elongation should not also be made the subject of specification. Tensile specimens represent, at best, only a small part of the cross-section of the rail. If taken from the center of the head, the results are affected by the segregation of the hardening elements, and do not represent the portion of the metal subjected to greatest strains in service; if taken from the exterior surface, the results are apt to be affected by the presence of a few partly welded blow holes. In neither case, therefore, can the tensile specimen be considered as fully representing the rail from which it was cut, much less any lot of rails from which the sample was selected.

(*e*) *Drop Tests Specified.*—All the foreign specifications reviewed, very properly contain a drop test; some of the drop tests specified, however, are open to considerable criticism. Where chemical composition is specified, and faithfully followed, the drop tests should not include a certain maximum deflection, but should simply specify that the piece of rail shall not break under a single blow on the head, the tup falling from a height increasing with the weight of the rail section. Where the engineer prefers to include a certain maximum deflection in the drop test, besides specifying that the piece of tested rail shall not fracture, the percentage of carbon should be omitted from the chemical requirements. Either method will insure a safe hard rail; but a specified range of 0.10% in carbon is perhaps a better and more prompt means of securing and maintaining uniformity in the product.

(*f*) *Dead-Weight Tests Specified.*—This transverse test under static load should include a determination of the yield point, and, by means of still heavier loads beyond the yield point, the determination of

greatest permanent deflection. The test may be looked upon as unnecessary when the chemical composition is fully specified and when the ability of the finished rails to withstand shock is determined by a drop test. However, the requirements of the dead-weight tests contained in foreign specifications are met without difficulty by American mills. Seventy per cent. of the forty-one specifications examined specify a dead-weight test, but in quite a number only a weight is used which will give no permanent set. The test is, therefore, frequently regarded in American mills as at least unnecessary, for, when the loads specified are within the elastic limit of the steel, the deflections are merely factors of the section of the rail. Mr. Colby.

5. Finish.

Some foreign specifications require, even after the templets have been approved by the engineer, resident abroad, that a section of the rail 12 ins. long must be submitted to the foreign engineer and approved in writing before the general rolling can proceed. This is a needless delay, as confidence can be safely placed in the skill and integrity of American manufacturers.

In a number of foreign specifications the permissible variations in height of sections are too rigidly drawn to allow for the unavoidable wear of the rolls. Smoothness of track is in nowise jeopardized by an allowance of $\frac{1}{64}$ in. under and $\frac{1}{32}$ in. over the specified height; they are the variations permitted by the best American railroads. A number of foreign specifications allow a less variation in length of rail than $\frac{1}{4}$ in., which is the uniform practice of American railroads.

Most foreign specifications allow a variation in weight of 1% for individual rails. This can be easily reduced to 0.50% for the entire order, but the rails should be paid for on actual, and not estimated, weights.

6. Inspection.

American mills are noted for their willingness to afford the inspector every facility to enable him to satisfy himself that the rails are being furnished in accordance with the specifications. In return, they have a right to expect from a foreign purchaser inspection which will not delay manufacturing operations. This is by no means always realized, and, as it is a frequent cause of expensive and annoying delays, it is a proper subject of comment.

The inspectors sent to American mills are frequently unfamiliar with rail inspection, and are, therefore, too strict in their inspection, and too slow in their decisions. Again, when the specifications require all four sides of every rail to be examined for flaws, all holes gauged, each rail tested for length and squared on each end, and stamped in four places by the inspector's mark, and also require him to be present

Mr. Colby. at the testing, to weigh 1% of the rails, and to watch the loading of the rails to see that only those bearing his stamp are loaded, it is manifestly unfair to the manufacturer to send but one inspector to the mill and expect the manufacturer to conform his product to the number of rails that one inspector can pass upon, in nine or ten hours of the twenty-four. Lack of inspectors frequently necessitates a change of rolls at the mills, and has delayed the departure of vessels on which the rails were being loaded. Assuredly, a sufficient number of inspectors to inspect the entire product of the mill each day should be furnished.

Some foreign specifications require that all rejected rails shall be piled up and kept until the completion of the contract, so that, at any time, the inspector may check them with the numbers entered in the book; and that no rejected rail shall be sold or broken until the completion of the contract. This is manifestly unfair to the manufacturer, because on a large contract, not continuously rolled, a considerable tonnage of second-quality rails might be tied up, which might otherwise be sold. Some foreign specifications state that the final acceptance of the rails and splice bars shall be at the port of delivery. This is unreasonable; final acceptance should be based on tests and inspection made at the place of manufacture.

The writer is confident that foreign purchasers have no desire to knowingly include in their specifications, requirements which, while securing no more efficient product, operate to limit competition and increase the cost of inspection and the price of product. This thought prompted the foregoing criticism of foreign rail specifications, which, it is hoped, will result to the mutual advantage of the foreign customer, and the American mills.

Mr. Webster. William R. Webster, M. Am. Soc. C. E.—From Mr. Hunt's discussion one might be led to believe that it was the usual practice to use high carbon steel in our rails. But such is not the case, as is shown by the fifteen American rail specifications given in the table prepared by Committee No. 1, of the American Section of the International Association. This table gives 0.45% to 0 55% carbon for the 100-lb. rails used by many of our railroads.

Most of the rails are rolled to the sections of this Society, the rails of 70 lbs. per yard and under giving better satisfaction than the heavier rails. It seems to the speaker that this difference in results is due more to the higher finishing temperature in rolling the heavier sections of rails than to any other cause. The large mass of metal in the head of the heavier rails carries the heat much longer than the thin metal in the flange, and the work of rolling has to be stopped on account of cold flange, when the metal in the head is still too hot. By putting more metal in the flange, to carry the heat, the head of the rail can be finished at a much lower temperature, and a closer grain

obtained. This is a matter which the Society could take up and investigate to advantage. Mr. Webster.

As a check on the finishing temperature in rolling, the speaker suggests that a limiting clause be put in the specifications as to the percentage of shrinkage which will be allowed in a rail, from its finishing temperature in rolling to the normal temperature. This amount of shrinkage in a 30-ft. rail is a good measure of the finishing temperature, and it only remains to prove by experiment just how much shrinkage it is safe to allow. Or, in other words, what finishing temperature should be used to produce the best results.

It has been claimed by some who take an extreme view of the present practice, that we are now "hardening our rails by the carbon and squirting them through the rolls." But, as a matter of fact, we know better to-day than ever before the true bearing of the chemical elements on the physical properties of the steel, and the change of structure due to the mechanical work of rolling or forging at the proper finishing temperature.

The beneficial effects of finishing a heavy rail at the proper temperature are shown in re-rolling heavy rails which have not given good service in use. This is accounted for by the annealing action of the furnace in heating the rails up to a low heat for rolling, thus breaking up the coarse grain, and not heating high enough to form it again, the final work of rolling on the head is at a low temperature, the flange being in a condition to allow this work at the proper temperature.

The report of the English Committee appointed by the Board of Trade "To Enquire into the Loss of Strength in Steel Rails through Use on Railways," will appear shortly, and give us much valuable information. It will, no doubt, be the true starting point for international rail specifications.

Many of our specifications, at the present time, have ear-marks in them showing the suggestion of the manufacturers in one section as against those of another, or suggestions of the makers of basic Bessemer steel as against those making acid Bessemer steel. What is wanted is to get rid of all this and take the whole matter up on its merits from an engineering and metallurgical standpoint. From the present indications it looks as though the International Association for Testing Materials would be invited by the Iron and Steel Institute to hold the next Congress in England in the fall of 1901. It is to be hoped that this will be brought about, as there is no society better fitted to assist in this great work than the Iron and Steel Institute of England.

The following is quoted from Appendix VII of the report of the English Committee:

"It is probable that the Board of Trade will expect or desire the Committee to recommend an analysis for the guidance of engineers or

Mr. Webster. manufacturers, which shall, in the judgment and experience of the Committee, be the composition of the steel to ensure a safe and good wearing rail. I therefore suggest, after careful consideration from the points of view of a manufacturer and user, that, exclusive of the iron, a steel rail should have the following range of composition:

	Minimum.		Maximum.
"Carbon	0.35	to	0.50
"Silicon	0.05	to	0.10
"Sulphur	0.04	to	0.08
"Phosphorus	....	to	0.08
"Manganese	0.75	to	1.00

"Mr. Edward P. Martin, of Dowlais, considered these suggested figures with me and approves them.

"(Signed) E. Windsor Richards."

On referring to the report of the American Committee No. 1, we find that the chemical limits in rails from 50 to 75 lbs. were as follows:

Carbon	0.35	to	0.50
Silicon	not over		0.20
Sulphur	.....		
Phosphorus	not over		0.10
Manganese	0.70	to	1.05

These recommendations are remarkably close, especially when one considers that the work was done in different countries by these Committees entirely independent of each other, and that in America only acid Bessemer steel is used for rails; while in England both the acid and basic Bessemer steel are used.

The results compared above indicate that the suggestions which have been made, that each country prepare standard or representative specifications for each class of material, can be easily carried out. It is not too much to expect that, at the General Congress of the International Association for Testing Materials, to be held in 1901, each country will present representative specifications, and that international specifications will be agreed upon.

The English Committee have been working at a great disadvantage, as in all cases of broken rails examined they knew nothing whatever of the heat treatment of these rails in rolling, and as this has such a great influence on the final structure of the rail, had it been known it would have accounted for many of the abnormal results referred to in the report. It is to be hoped that the work of the Committee of the Board of Trade will be continued, that rails will be rolled under known conditions, and that rails from the same heat of steel will be finished hot, medium and cold, in order to get at the true value of the mechanical work of rolling at different temperatures. This will give valuable information in one of the most important lines of research, which has been too much neglected up to the present time.

JOHN F. WALLACE, President, Am. Soc. C. E.—Two of the American engineers who have spoken on this question represent inspecting consulting engineers, and one the manufacturers. The speaker would like to say one word from the purchasers' side. The railroads themselves, up to date, have had very little to say about the specifications of the rails used in America, and mainly on account of business reasons. The railways to the east of the Allegheny Mountains have naturally been confined to the use of rails manufactured in that district, first, on account of the fact that the manufacturers were also the customers of the railroads, and produced a great deal of freight and traffic; and secondly, because they were nearer the point of consumption. The same controlling elements exist in the mills in the vicinity of Chicago. The roads of the West, tributary to Chicago, were naturally inclined to purchase rails from the Chicago mills, and the result has been that the roads east of the Allegheny Mountains have been compelled to use rails manufactured in their district, which could be manufactured from the ores east of that point; and the roads in the Mississippi Valley and the West have generally been confined to the use of the rails manufactured in Chicago. A great many roads have endeavored to enforce their own specifications. On the other hand, the mills have endeavored to require the railroads to accept their specifications, coupled with a guarantee that any rails affected during the period of 5 years should be replaced. Mr. Wallace.

The railroads, to-day, through their engineers and their own special organization, which is looking after this particular matter, are now taking up the question of adopting specifications and asking the manufacturers to conform to them. The matter is now in the hands of a Committee of Engineers, who do not propose to draw up specifications which are impossible to fill; but intend to find out, from the character of the ores in different parts of the country, and from the processes of manufacture, what are the possibilities, and will endeavor to decide on specifications which are practical and common-sense. In the formation of such specifications, however, they expect to consult with the manufacturers and with the inspecting engineers.

In reference to phosphorus, some years ago on the Illinois Central Railroad, on a section of road where the traffic was exceedingly heavy, there was in use a 60-lb. rail, laid in 1878. Twenty years after that rail was laid, it was removed in order to replace it with an 85-lb. rail. The rail gave such extraordinarily good service that it was analyzed and found to run as high as 0.012 and 0.015 phosphorus—from 50 to 100% more than the specifications were then calling for.

SIR LOWTHIAN BELL, M. Inst. C. E.—There are probably few questions connected with the manufacture and behavior of rails while in use which demand a more extended area of examination than those raised by Mr. Hunt. The opinions about to be given are those gathered from many years' experience as a manufacturer supple- Sir Lowthian Bell

Sir Lowthian Bell.

TABLE No. 1.—Results of Tests made at Park Lane Stores, Gates-Middle and Bottom of Ingots, as per Sir Lowthian Bell's, 3 ft. apart.

Distinguishing No. of rail.....	991		992		993		994		995		996		997		998	
Ingot No.....	1		1		1		2		2		2		3		3	
Part of ingot taken from.....	Top.		Middle.		Bottom.		Top.		Middle.		Bottom.		Top.		Middle.	
Results of tests..	Fall ft	Def. ins.	Fall ft	Def. ins.	Fall ft	Def. ins.	Fall ft	Def. ins.	Fall ft	Def. ins.	Fall ft	Def. ins.	Fall ft	Def. ins.	Fall ft	Def. ins.
	5	$\frac{5}{8}$	5	$\frac{11}{16}$	5	$\frac{5}{8}$	5	$\frac{11}{16}$	5	$\frac{5}{8}$	5	$\frac{11}{16}$	5	$\frac{11}{16}$	5	$\frac{11}{16}$
	5	$1\frac{1}{8}$	5	$1\frac{5}{16}$	5	$1\frac{3}{16}$	5	$1\frac{1}{4}$	5	$1\frac{5}{16}$	5	$1\frac{5}{8}$	5	$1\frac{5}{16}$	5	$1\frac{5}{16}$
	7	$1\frac{3}{4}$	7	$1\frac{1}{2}$	7	$1\frac{15}{16}$	7	broke	7	$1\frac{1}{2}$	7	$1\frac{3}{4}$	7	$1\frac{15}{16}$	7	$1\frac{15}{16}$
	10	$2\frac{3}{4}$	10	$2\frac{11}{16}$	10	$2\frac{3}{4}$	..	.....	10	$2\frac{11}{16}$	10	$2\frac{5}{8}$	10	$2\frac{3}{4}$	10	$2\frac{3}{4}$
	15	$4\frac{3}{16}$	15	4	15	$4\frac{3}{16}$	..	.....	15	$4\frac{1}{8}$	15	$4\frac{1}{16}$	15	$4\frac{1}{8}$	15	$4\frac{3}{16}$
	21	broke	21	$6\frac{1}{2}$	21	$6\frac{7}{16}$	..	.....	21	broke	21	$6\frac{1}{2}$	21	broke	21	broke
	..	.....	27	broke	27	broke	..	.....	..	.....	27	$9\frac{15}{16}$	..	.....	..	.....
	..	.....	..	.....	..	.....	..	.....	..	.....	27	broke	..	.....	..	.....
Sum of falling feet (breaking blow halved)...	$52\frac{1}{2}$		$76\frac{1}{2}$		$76\frac{1}{2}$		$13\frac{1}{2}$		$52\frac{1}{2}$		$103\frac{1}{2}$		$52\frac{1}{2}$		$52\frac{1}{2}$	

Note.—These samples taken from

Composition—								
Phosphorus....	0.043	0.048	0.043	0.040	0.044	0.053	0.043	0.050
Sulphur.....	0.050	0.060	0.070	0.050	0.060	0.060	0.070	0.060
Carbon.....	0.560	0.560	0.570	0.500	0.510	0.550	0.560	0.560
Silicon.....	0.069	0.066	0.057	0.072	0.072	0.084	0.081	0.069
Manganese	0.950	0.960	0.970	1.020	1.030	1.030	0.950	1.060
Arsenic.....	0.061	0.065	0.061	0.028	0.018	0.044	0.049	0.032
Iron by difference.....	98.267	98.241	98.229	98.290	98.266	98.179	98.247	98.169

The weight of the rail was 90 lbs. per yard, the length tested

mented by a still longer experience as a director of one of the most important railroads in the United Kingdom. A life so spent seems to embrace all the conditions essential to the present discussion.

It may be well here to describe the mode of testing used in ascertaining the resistance to fracture offered by the rails under examination:

A weight of one ton (2 240 lbs.), and capable of being detached at any point of its travel, was raised by steam power between two girders of considerable height. The usual plan was to raise the weight a very few feet at first, and to increase the height of fall gradually, until fracture was effected. No account was taken of the acceleration of the fall, and therefore of the power, by the continued operation of gravita-

HEAD, ON 18 PIECES OF NEW "HEMATITE" STEEL RAILS, FROM TOP, INSTRUCTIONS. TESTED JUNE 20TH, 1898; BALL, 1 TON; BEARINGS,

999		1000		1001		1002		1003		1004		1005		1006		1007		1008	
3		4		4		4		5		5		5		6		6		6	
Bottom.		Top.		Middle.		Bottom.		Top.		Middle.		Bottom.		Top.		Middle.		Bottom.	
Fall	Def.	Fall	Def.	Fall	Def.	Fall	Def.	Fall	Def.	Fall	Def.	Fall	Def.	Fall	Def	Fall	Def.	Fall	Def.
ft	ins.	ft	ins.	ft	ins.	ft	ins.	ft	ins.	ft	ins.	ft	ins.	ft	ins.	ft	ins.	ft	ins.
5	1 1/16	5	1 1/16	5	1 1/16	5	5/8	5	1 1/16	5	1 5/8	5	1 1/16	5	1 1/16	5	1 5/8	5	5/8
5	1 1/8	5	1 1/16	5	1 5/16	5	1 1/8	5	1 5/16	5	1 3/16	5	1 1/8	5	1 5/16	5	1 3/4	5	1 1/16
7	1 1/4	7	1 1/8	7	1 3/16	7	1 3/16	7	1 1/16	7	1 3/16	7	1 1/4	7	1 1/8	7	1 3/4	7	1 1/16
10	2 5/16	10	broke	10	2 1/4	10	2 1/4	10	2 1/4	10	2 1/16	10	2 5/8	10	2 1/16	10	2 1/16	10	2 1/16
15	4 1/16	..	.....	15	4 1/16	15	4 1/16	15	4 1/16	15	4 1/2	15	4 1/2	15	4 1/2	15	4 1/2	15	3 15/16
21	broke	..	.....	21	6 7/16	21	6 1/2	21	6 5/8	21	6 1/2	21	6 5/8	21	6 1/2	21	broke	21	6
..	.....	..	.....	27	broke	27	broke	27	broke	27	broke	27	broke	27	broke	..	.....	27	broke
..	.....	..	.....	..	.....	..	.....	..	.....	..	.....	..	.....	..	.....	..	.....	..	.....
52½		22		76½		76½		76½		76½		76½		76½		52½		76½	

six ingots, three out of each.

0.051	0.059	0.073	0.070	0.060	0.060	0.059	0.057	0.070	0.062
0.060	0.060	0.060	0.060	0.050	0.060	0.050	0.040	0.050	0.050
0.560	0.550	0.570	0.560	0.560	0.580	0.570	0.500	0.550	0.580
0.069	0.066	0.072	0.063	0.069	0.066	0.063	0.072	0.087	0.084
1.050	1.060	1.070	1.040	1.040	1.070	1.030	0.870	0.930	1.060
0.045	0.051	0.026	0.039	0.061	0.059	0.045	0.049	0.057	0.063
98.165	98.154	98.129	98.168	98.160	98.105	98.183	98.412	98.256	98.101

was 10 ft., and in each case the running head was up.

tion, because this did not exhaust by any means the disturbing causes of the calculation. The chief of these is the unknown absorption of power by the deflection of the rail at each blow. The plan followed was to sum up the various falls and count the last at one-half its real amount. The estimate, therefore, is one of comparison only.

In evidence of irregularity in results the case of the trials of bull-headed steel rails of 90 lbs. per yard may be given. One of these gave way under a united fall of 23 ft., while a second one required for its fracture 199 ft. Chemical analyses of several specimens were made without throwing any light on this very great difference in the fall required for rupture. This will be best reserved for cases to be presently described.

Sir Lowthian Bell.

Those members who have read the classic work of Professor Henry Marion Howe, who confirmed observations made at the Clarence Works, will remember his description of the absorbing power of liquid steel in occluding gaseous matter, which latter was given off as the molten metal cooled. He naturally inferred that the uppermost portion of the ingot would contain the largest portion of the liberated gas which was imprisoned in spherical cells. When the ingot was rolled into a rail these cells were flattened but not entirely obliterated.

Trials were made by dividing each ingot into three parts, which were marked top, middle and bottom, and a rail was rolled from each. Six rails from each section were rolled and were exposed to the following test. A reference to Table No. 1 shows that the fractures took place as follows:

From the "Tops" at an average of 48.9 ft., "Middles" 64.5 ft., and the "Bottoms" 77 ft., and, further, that the analyses fail to connect these with any differences in chemical composition.

A committee appointed by the British Board of Trade has concluded a long inquiry, in which evidence of an exhaustive character was given. In it the data just quoted, besides much more of a highly important character, are to be found.

The changes in molecular structure, supplemented by microphotographic plates, are described at great length and with great ability by Professor Roberts Austen.

Mr. Smelt.

J. D. Smelt, M. Inst. C. E.—The speaker has had occasion recently to advise the Argentine Great Western Railway Company on the relative value of American and British tenders for rails.

The section tendered upon, 72.8 lbs. per yard, was designed by the speaker in 1893, and has been rolled several times since that date in Great Britain and also in the United States. It is shown in full black lines in Fig. 1. The enquiry was issued for rails to be manufactured to the speaker's specification. One of the leading American firms tendered in accordance with the Argentine Great Western section, but in accordance with their standard specification for quality. The differences between the two specifications are shown in the table on Fig. 1, the chief differences being the high phosphorus and silicon of the American specification and the lower tensile strength and elongation, coupled with the somewhat inconsistently easy falling-weight test.

The price per ton quoted by the American firm was considerably lower than the British quotations, and the principal questions to decide were: First, what is the relative durability of 43-ton steel as compared with 38-ton steel; and, secondly, the amount of attendant risk due to the presence of high phosphorus and silicon, which, in a rail with a somewhat thin flange, might be considerable.

With regard to durability, it appeared to be approximately in pro-

Chemical Composition	Argentine Gt. Western	American
Carbon	0.4 to 0.5	0.4 to 0.5
Silicon	up to 0.08	up to 0.2
Sulphur	up to 0.06	up to 0.05
Phosphorus	up to 0.06	up to 0.10
Manganese	up to 1.0	0.7 to 0.9
Falling Weight Test on 3-Ft. Bearings		
Weight of Tup	2240 lbs.	2000 lbs.
Height of Fall	13 ft.	17 ft.
Number of Blows (without fracture)	4	1
Deflection under 1st Blow	2 to 3 ins.	
" " 2d "	4 to 5 ins.	
Tensile Test		
Tensile Strength (Tons per square inch)	43	38
Elongation	12% in 10 ins.	12% in 2 ins.
Contraction of Area	20%	

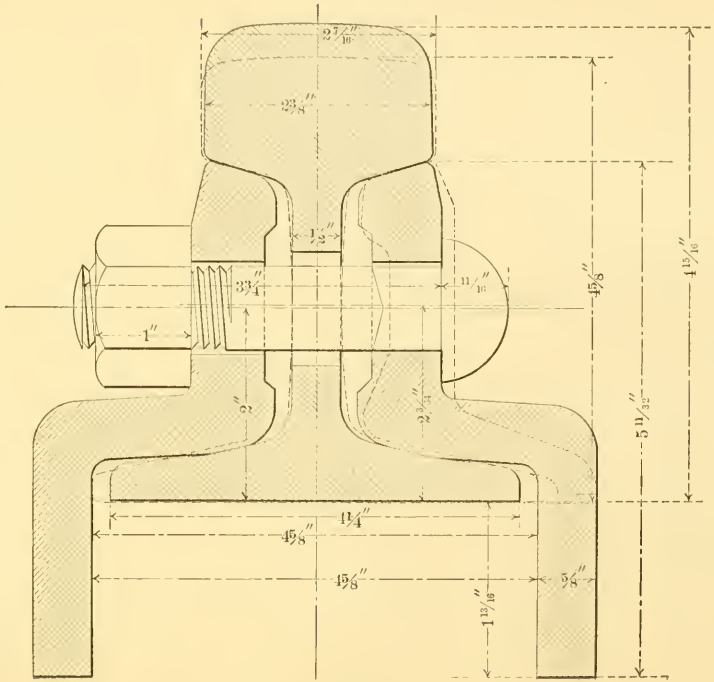

Fig. 1.

Mr. Smelt. portion to the tensile strength, which is, in the speaker's opinion, independent of the means adopted to produce it; that is to say, it matters little whether the high or low tensile strength is due to the difference in temperature of the material during rolling, or to difference in chemical composition. Perhaps Mr. Hunt and other members can give some practical information on the subject.

Another point, to which attention should be directed, is the difference in the American standard sections of 70-lb. rails and fish-plates (shown in dotted lines on Fig. 1), as compared with the sections designed for the Argentine Great Western Railway.

In the Argentine Great Western rail, the speaker allowed for a wear of $\frac{5}{8}$ in., and finds that the depth of the head of the American standard is about $\frac{5}{16}$ in. less, which would appear to indicate about half the life, apart from the difference in tensile strength.

With regard to fish-plates, the Argentine Great Western deep section shown on Fig. 1 was designed to make the joint of equivalent strength to the solid rail when worn, and the results of tests prove the design to be correct in this respect.

The American fish-plates of angular design cannot, therefore, be of the same relative strength, especially when taking into account the very soft material of which they are made.

TRANSACTIONS

OF THE

American Society of Civil Engineers.

INDEX.

VOLUME XLIV.

DECEMBER, 1900.

NOTE.—In this Index, Subjects and Writers are given Alphabetically. Titles of Papers are enclosed by quotation marks when indexed alone or under the Author's name. When indexed under the name of a person discussing a paper, the titles are not so enclosed. Titles are entered with reference to the subject and also under the first significant word.

INDEX.

VOLUME XLIV.

DECEMBER, 1900.

ALGÆ.
——in Pipes, 55, 59, 68.

ALLEN, Kenneth. Topographic Surveys, 113.

AVERILL, F. L. Coal Pockets, 143.

BELL, Sir Lowthian. Recent Practice in Rails, 499.

BOECKLIN, Werner, Jr. Coal Conveyor, 146.

BOGGS, J. I. Preservation of Timber, 203.

BOGUE, V. G. Preservation of Timber, 198.

BOLTON, Reginald Pelham. High Buildings, 454.

BUCK, R. S. Topographic Surveys, 105.

BUILDINGS.
Height of— An Informal Discussion, 449.

CARR, Albert. Power Station of the Metropolitan Street Railway Company, 144.

CARTWRIGHT, Robert. Preservation of Timber, 198.

CHIMNEYS.
Power Station of the Metropolitan Street Railway Company. L. G. Montony, 124; W. A. Low, 145.

CHRISTIAN, G. L. Topographic Surveys, 115.

COAL CONVEYORS.
L. G. Montony, 137; W. A. Low, 146; Werner Boecklin, Jr., 146; J. D. Kent, 147.

COAL POCKETS.
L. G. Montony, 128, 147; F. L. Averill, 143; W. A. Low, 146.

COLBY, Albert Ladd. Recent Practice in Rails, 491.

CURTIS, Walter W. Preservation of Timber, 204.

DAMS.
"On the Flow of Water Over—" George W. Rafter. (With Discussion.) 220.

DAVEY, Henry. Filtration of Water, 419.

DEACON, George F. Filtration of Water, 405.

DEGHUEE, J. H. Preservation of Timber, 195.

DUTY OF WATER.
"Irrigation Studies." Elwood Mead. (With Discussion.) 149.

ELECTRIC RAILROADS.
"The Ninety-Sixth Street Power Station of the Metropolitan Street Railway Company, of New York City." L. G. Montony. (With Discussion.) 119.

"EXPERIMENTS ON THE FLOW OF WATER IN THE SIX-FOOT Steel and Wood Pipe Line of the Pioneer Electric Power Company, at Ogden, Utah. Second Series." Charles D. Marx, Charles B. Wing and Leander M. Hoskins, 34.
Discussion: E. Kuichling, 55; G. C. Whipple, 59; J. C. Meem, 60; G. S. Williams, 61; George W. Rafter, 67; George H. Fenkell, 73; D. C. Henny, 80; Charles W. Sherman, 84; Desmond Fitz-Gerald, 86; Charles D. Marx, Charles B. Wing and Leander M. Hoskins, 88.

FELT, C. F. W. Preservation of Timber, 210.

FENKELL, George H. Flow of Water in Pipes, 73.

"FILTRATION OF WATER FOR PUBLIC USE." An Informal Discussion. Rudolph Hering, 399; George F. Deacon, 405; George W. Fuller, 408; Samuel Rideal, 417; Henry Davey, 419; J. N. Greene, 420; Ad. Kemna, 420; K. E. Hilgard, 425; Walter Hunter, 430; Philip H. Palmer, 431; E. Perrett, 433; Andrew Johnston, 433; H. Alfred Roechling, 436; Nicholas Simin, 438; Rudolph Hering, 443.

FISHER, Wager. Topographic Surveys, 114.

FITZGERALD, Desmond.
Flow of Water in Pipes, 86.
Algæ and Spongilla in Pipes, 86.

FLOW OF WATER. See "*Water, Flow of.*"

FOOTE, A. D. Weir Registers, 177.

FUERTES, E. A. Flow of Water over Dams, 339.

FULLER, George W. Filtration of Water, 408.

FURBER, William Copeland. High Buildings, 472.

GOLDMARK, Henry.
Topographic Surveys, 105.
Preservation of Timber, 197.

GREENE, J. N. Filtration of Water, 420.

"HEIGHT OF BUILDINGS." An Informal Discussion. Corydon T. Purdy, 449; Reginald Pelham Bolton, 454; Robert W. Hunt, 457; W. E. Riley, 458; H. de B. Parsons, 462; R. J. Gifford Read, 468; Oscar J. Kirby, 468; John F. Wallace, 468; Maurice M. Sloan, 470; William Copeland Furber, 472.

HENNY, D. C. Flow of Water in Pipes, 80.

HERING, Rudolph.
Irrigation and Drainage, 176.
Filtration of Water, 399, 443.

HILGARD, K. E. Filtration of Water, 425.

HIMES, A. J. Topographic Surveys, 105.

"HISTORY OF THE PENNSYLVANIA AVENUE SUBWAY, Philadelphia, and Sewer Construction Connected Therewith." George S. Webster and Samuel Tobias Wagner, 1.
Discussion: George S. Webster and Samuel Tobias Wagner, 31.

HORTON, Robert E. Flow of Water over Dams, 340.

HOSKINS, Leander M. "Experiments on the Flow of Water in the Six-Foot Steel and Wood Pipe Line of the Pioneer Electric Power Company, at Ogden, Utah. Second Series," 34, 88.

HOWE, Horace J. Preservation of Timber, 201.

HUNT, Robert W.
High Buildings, 457.
Recent Practice in Rails, 475.

HUNTER, Walter. Filtration of Water, 430.

HYDRAULICS.
"Experiments on the Flow of Water in the Six-Foot Steel and Wood Pipe Line of the Pioneer Electric Power Company, at Ogden, Utah. Second Series." C. D. Marx, C. B. Wing and L. M. Hoskins. (With Discussion.) 34.
"Irrigation Studies." Elwood Mead. (With Discussion.) 149.
"On the Flow of Water over Dams." George W. Rafter. (With Discussion.) 220.

"IRRIGATION STUDIES." Elwood Mead, 149.
Discussion: Mansfield Merriman, 176; Rudolph Hering, 176; A. D. Foote, 177; Elwood Mead, 179.

JOHNSTON, Andrew. Filtration of Water, 433.

KEMNA, Ad. Filtration of Water, 420.

KENT, J. D. Coal Conveyor, 147.

KIRBY, Oscar J. High Buildings, 468.

KUICHLING, EMIL.
Flow of Water in Pipes, 55.
Tuberculation in Pipes, 55.
Algæ and Spongilla in Pipes, 55.
Loss of Head in Venturi Meters, 59.
Flow of Water over Dams, 335.

KUMMER, F. A. "A Proposed Method for the Preservation of Timber." 181, 211.

LANDRETH, WILLIAM B. "Recent Stadia Topographic Surveys: Notes Relating to Methods and Cost." 92, 117.

LOW, EMILE. Topographic Surveys, 100.

LOW, W. A. Power Station of the Metropolitan Street Railway Company, of New York City, 145.

MACGREGOR, R. A. Topographic Surveys, 112.

MAGOR, HENRY B. Topographic Surveys, 103.

MANOMETERS.
"Experiments on the Flow of Water in the Six-Foot Steel and Wood Pipe Line of the Pioneer Electric Power Company, at Ogden, Utah. Second Series." C. D. Marx, C. B. Wing and L. M. Hoskins. (With Discussion.) 34.

MARX, CHARLES D. "Experiments on the Flow of Water in the Six-Foot Steel and Wood Pipe Line of the Pioneer Electric Power Company, at Ogden, Utah. Second Series." 34, 88.

MEAD, ELWOOD. "Irrigation Studies," 149, 179.

MEEM, J. C. Flow of Water in Pipes, 60.

MERRIMAN, MANSFIELD. Irrigation Studies, 176.

METERS.
Venturi— "Experiments on the Flow of Water in the Six-Foot Steel and Wood Pipe Line of the Pioneer Electric Power Company, at Ogden, Utah. Second Series." C. D. Marx, C. B. Wing and L. M. Hoskins. (With Discussion.) 34.
Loss of head in Venturi—59, 90.
(*See Weir Registers.*)

MONCURE, WILLIAM. Preservation of Timber, 202.

MONTFORT, R. Preservation of Timber, 208.

MONTONY, L. G. "The Ninety-sixth Street Power Station of the Metropolitan Street Railway Company, of New York City." 119, 147.

NELLES, GEORGE T. Flow of Water over Dams, 359.

"NINETY-SIXTH STREET POWER STATION OF THE METROpolitan Street Railway Company, of New York City." L. G. Montony, 119.

Discussion: F. L. Averill, 143; Albert Carr, 144; W. A. Low, 145; Werner Boecklin, Jr., 146; J. D. Kent, 147; L. G. Montony, 147.

"ON THE FLOW OF WATER OVER DAMS." George W. Rafter, 220.

Discussion: George Y. Wisner, 315; Gardner S. Williams, 316; E. Kuichling, 335; E. A. Fuertes, 339; Robert E. Horton, 340; Walter C. Parmley, 346; George T. Nelles, 359; George W. Rafter, 383.

OSGOOD, Joseph O. Preservation of Timber, 198.

PALMER, Philip H. Filtration of Water, 431.

PARMLEY, Walter C. Flow of Water over Dams, 346.

PARSONS, H. de B. High Buildings, 462.

PERCOLATION.

"Irrigation Studies." Elwood Mead. (With Discussion.) 149.

PERRETT, E. Filtration of Water, 433.

PIEZOMETERS.

"Experiments on the Flow of Water in the Six-Foot Steel and Wood Pipe Line of the Pioneer Electric Power Company, at Ogden, Utah. Second Series." C. D. Marx, C. B. Wing and L. M. Hoskins. (With Discussion.) 34.

"On the Flow of Water over Dams." George W. Rafter. (With Discussion.) 220.

PIPE.

"Experiments on the Flow of Water in the Six-Foot Steel and Wood—Line of the Pioneer Electric Power Company, at Ogden, Utah. Second Series." C. D. Marx, C. B. Wing and L. M. Hoskins. (With Discussion.) 34.

POWER PLANT.

"The Ninety-sixth Street Power Station of the Metropolitan Street Railway Company, of New York City." L. G. Montony. (With Discussion.) 119.

"PROPOSED METHOD FOR THE PRESERVATION OF TIMber." F. A. Kummer. 181,

Discussion: Henry G. Prout, 191; J. H. Deghuée, 195; F. W. Skinner, 195; Henry Goldmark, 197; Robert Cartwright, 198; V. G. Bogue, 198; Joseph O. Osgood, 198; Samuel Whinery, 200; Horace J. Howe, 201; William Moncure, 202; J. I. Boggs, 203; Walter W. Curtis, 204; R. Montfort, 208; C. F. W. Felt, 210; George S. Valentine, 210; F. A. Kummer, 211.

PROUT, Henry G. Preservation of Timber, 194.

PURDY, Corydon T. High Buildings, 449.

RAFTER, George W.
Flow of Water in Pipes, 67.
Algæ and Spongilla, 68.
Topographic Surveys, 116.
"On the Flow of Water over Dams." 220, 383.

RAILROADS.
Pennsylvania Avenue Subway, Philadelphia. George S. Webster and Samuel Tobias Wagner, 1.

RAILROADS, TRACK.
"A Proposed Method for the Preservation of Timber." F. A. Kummer. (With Discussion.) 181.

RAILS.
"Recent Practice in—" An Informal Discussion, 475.

RAILWAYS, STREET.
"The Ninety-sixth Street Power Station of the Metropolitan Street Railway Company, of New York City." L. G. Montony. (With Discussion.) 119.

READ, R. J. Gifford. High Buildings, 468.

"RECENT PRACTICE IN RAILS." An Informal Discussion. Robert W. Hunt, 475; Albert Ladd Colby, 491; William R. Webster, 496; Sir Lowthian Bell, 499; J. D. Smelt, 502.

"RECENT STADIA TOPOGRAPHIC SURVEYS: NOTES RELATing to Methods and Cost." William B. Landreth, 92.
Discussion: Emile Low, 100; John F. Wallace, 103; Henry B. Magor, 103; R. S. Buck, 105; Henry Goldmark, 105; A. J. Himes, 105; R. A. MacGregor, 112; Kenneth Allen, 113; Wager Fisher, 114; G. L. Christian, 115; George W. Rafter, 116; William B. Landreth, 117.

RIDEAL, Samuel. Filtration of Water, 417.

RILEY, W. E. High Buildings, 458.

ROECHLING, H. Alfred. Filtration of Water, 436.

SEWERAGE.
"Pennsylvania Avenue Subway, Philadelphia, and Sewer Construction Connected Therewith." George S. Webster and Samuel Tobias Wagner. (With Discussion.) 1.

SHERMAN, Charles W.
Flow of Water in Pipes, 84.
Algæ and Spongilla in Pipes, 84.

SIMIN, Nicholas. Filtration of Water, 438.

SKINNER, F. W. Preservation of Timber, 195.

SLOAN, Maurice M. High Buildings, 470.

SMELT, J. D. Recent Practice in Rails, 502.

SPONGILLA.
—In Water Pipes, 55, 59, 68.
STREAMS, FLOW OF.
"Irrigation Studies." Elwood Mead. (With Discussion.) 149.
SUBWAYS.
Pennsylvania Avenue— Philadelphia. George S. Webster and Samuel Tobias Wagner, 1.
SURVEYING INSTRUMENTS.
"Recent Stadia Topographic Surveys: Notes Relating to Methods and Cost." W. B. Landreth. (With Discussion.) 92.
SURVEYS.
"Recent Stadia Topographic— Notes Relating to Methods and Cost." W. B. Landreth. (With Discussion.) 92.
TIES.
"A Proposed Method for the Preservation of Timber." F. A. Kummer. (With Discussion.) 181.
TIMBER, PRESERVATION OF.
"A Proposed Method for the Preservation of—" F. A. Kummer. (With Discussion.) 181.
TOPOGRAPHY.
"Recent Stadia Topographic Surveys: Notes Relating to Methods and Cost." W. B. Landreth. (With Discussion.) 92.
TUBERCULATION IN PIPES.
—of the Rochester Water-Works, 55.
TUNNELS.
"Pennsylvania Avenue Subway, Philadelphia, and Sewer Construction Connected Therewith." George S. Webster and Samuel Tobias Wagner. (With Discussion.) 1.
VALENTINE, George S. Preservation of Timber, 210.
WAGNER, Samuel Tobias. "History of the Pennsylvania Avenue Subway, Philadelphia, and Sewer Construction Connected Therewith." 1, 31.
WALLACE, JOHN F.
Topographic Surveys, 103.
High Buildings, 468.
WATER, FLOW OF.
"Experiments on the Flow of Water in the Six-Foot Steel and Wood Pipe Line of the Pioneer Electric Power Company, at Ogden, Utah. Second Series." C. D. Marx, C. B. Wing and L. M. Hoskins. (With Discussion.) 34.
"Irrigation Studies." Elwood Mead. (With Discussion.) 149.
"On the Flow of Water over Dams." George W. Rafter. (With Discussion.) 220.

WATER SUPPLY.
"Filtration of Water for Public Use." An Informal Discussion, 399.

WEBSTER, GEORGE S. "History of the Pennsylvania Avenue Subway, Philadelphia, and Sewer Construction Connected Therewith." 1, 31.

WEBSTER, WILLIAM R. Recent Practice in Rails, 496.

WEIR REGISTERS.
"Irrigation Studies." Elwood Mead. (With Discussion.) 149.

WEIRS.
"On the Flow of Water over Dams." George W. Rafter. (With Discussion.) 220.

WHINERY, SAMUEL. Preservation of Timber, 200.

WHIPPLE, G. C. Algæ, Spongilla, etc., in Pipes, 59.

WILLIAMS, GARDNER S.
Flow of Water in Pipes, 61.
Flow of Water over Dams, 316.

WING, CHARLES B. "Experiments on the Flow of Water in the Six-Foot Steel and Wood Pipe Line of the Pioneer Electric Power Company, at Ogden, Utah. Second Series." 34, 88.

WISNER, GEORGE Y. Flow of Water over Dams, 315.

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